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MUNICIPAL

MASTER DRAINAGE PLAN

HAMILTON BEACH

Prepared for

CITY OF HAMILTON

July 1999

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Monaghan
CONSULTING ENGINEERS • SURVEYORS • PLANNERS

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July 29, 1999
File: 14-99015-01-I01

City of Hamilton
City Hall
71 Main Street West
Hamilton, Ontario
L8P 4Y5

Attention: **Ms. Joanne Hickey-Evans**
Manager of Policy and Neighbourhood Planning

Dear Ms. Hickey-Evans,

Re: **Submission of Master Drainage Plan for Hamilton Beach**

We are pleased to submit ten copies of the Master Drainage Plan for Hamilton Beach. This presents the results of our study and provides recommendations to mitigate current street flooding and lot grading for lots to be re-developed.

We trust you will find our submission to be satisfactory but if you have any questions please call.

Yours truly,
MARSHALL MACKLIN MONAGHAN LIMITED

Jennifer Payne, M.Sc., P.Eng.
Senior Project Engineer
Water Resources Department

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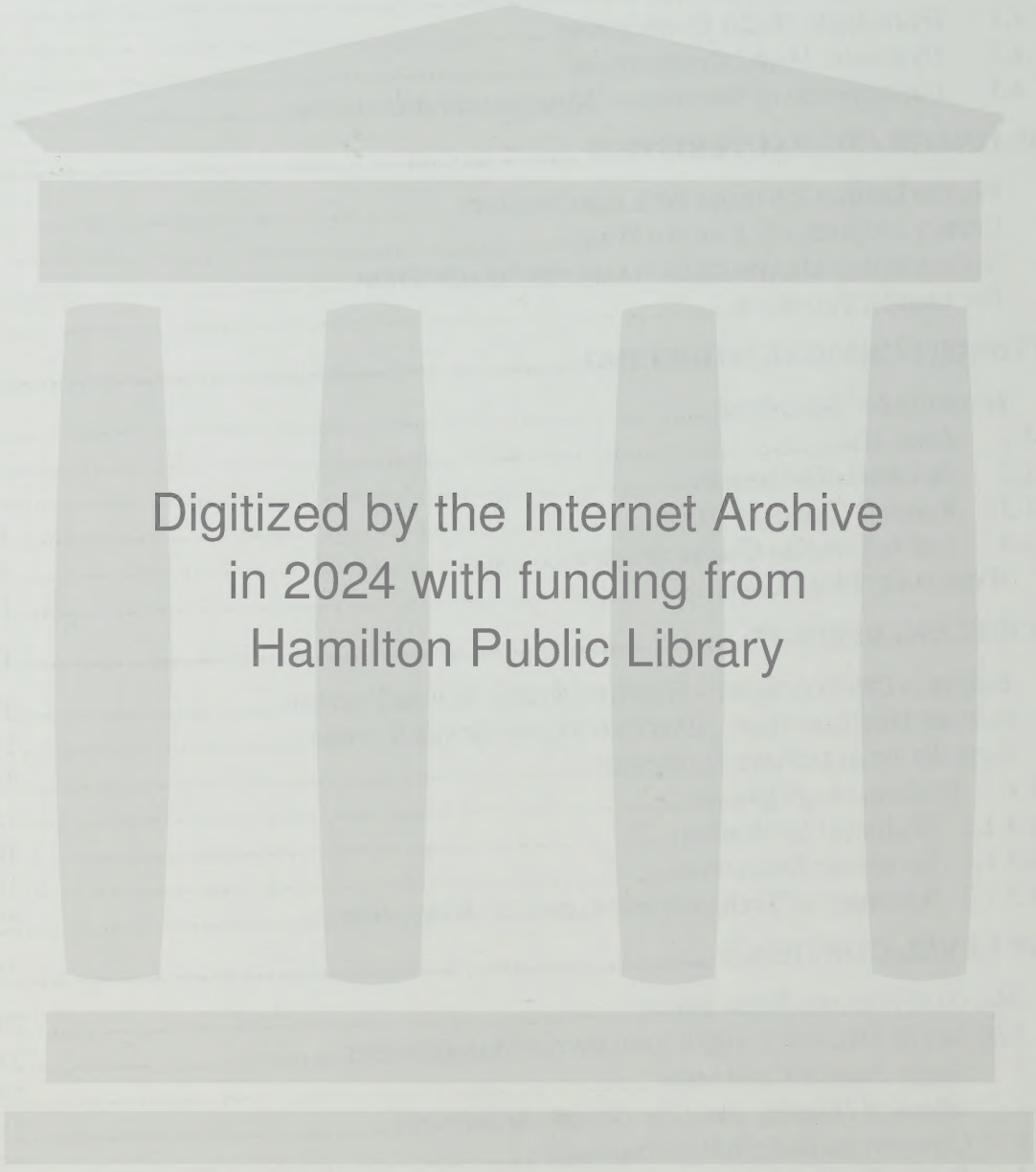
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1 INTRODUCTION

1.1 Site Location and Description

The Hamilton Beach neighbourhood is located on part of the narrow 6.5 km strip of land that forms a natural bridge between the two sides of Hamilton Harbour (Figure 1.1). The strip of land also acts as a barrier between the Harbour and Lake Ontario except for the Burlington canal that marks the division between the City of Hamilton and the City of Burlington.

1.1.1 Brief History of the Hamilton Beach Neighbourhood

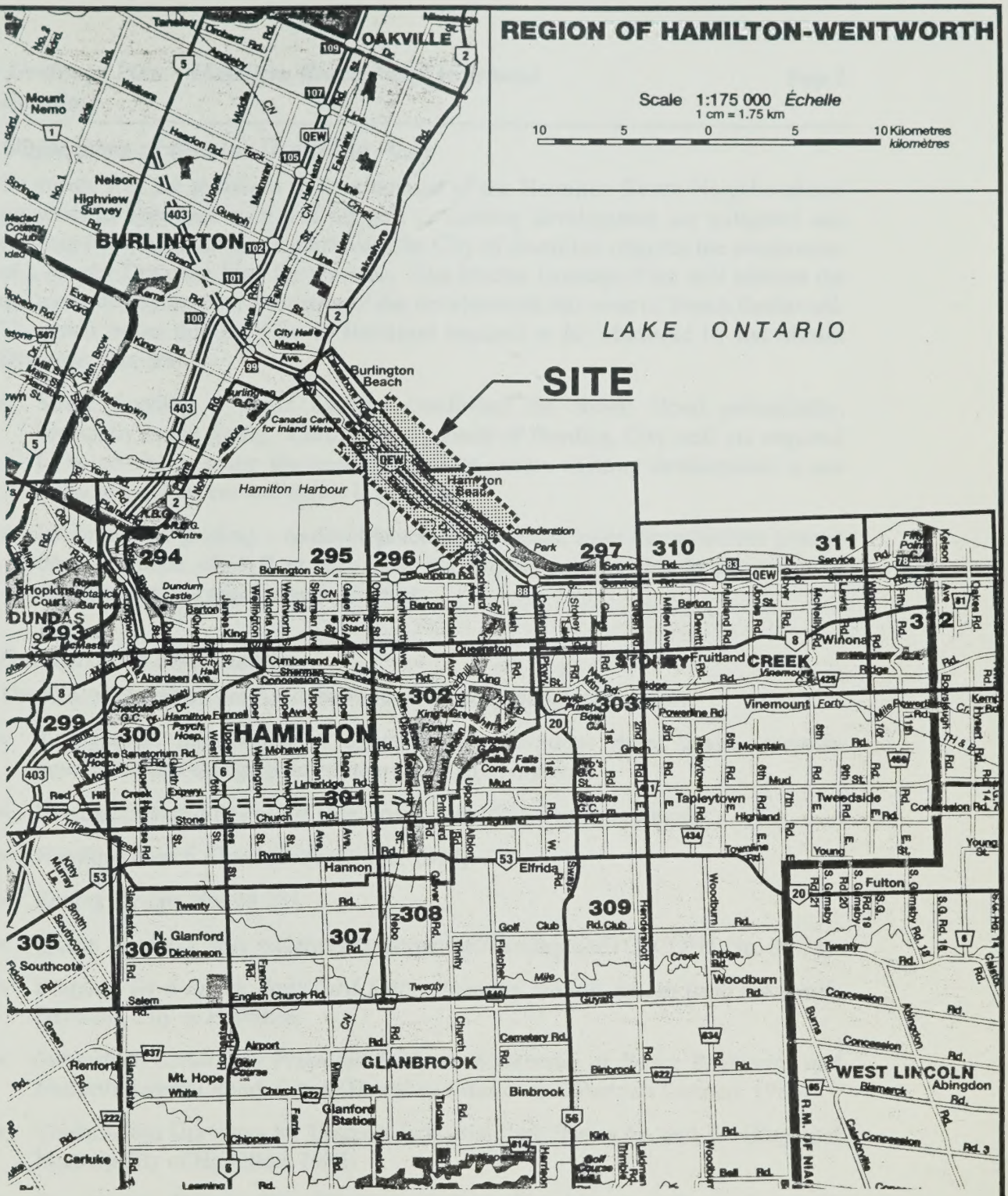
Historically the Hamilton Beach neighbourhood was developed as a shipping and summer resort destination. By the early 1900s the popularity of the beach as a summer resort declined. The Beach continued to grow as a community with the construction of permanent residences rising. As the number of permanent residences increased the property damage due to flooding increased as well.

In 1933 the Queen Elizabeth Way was constructed across the Hamilton Beach neighbourhood; a swing bridge crossed the canal. In 1954 construction began on the Burlington Bay Skyway Bridge. This construction was initiated since the original bridge crossing the canal had been damaged beyond repair and the volume of traffic crossing the Beach Neighbourhood was becoming intolerable to the Beach Neighbourhood. The Skyway Bridge was finally opened to traffic in November 1958.

In the early 1970s flooding had reached a critical point. The City of Hamilton received provincial and federal support to purchase property from anyone who wanted to sell on the Hamilton Beach neighbourhood. Numerous properties were purchased and the houses demolished. The intent was that the neighbourhood would be turned into parkland within 20 years. By the late 1970s the Hamilton Beach Preservation Committee was formed. Their aim was to improve communications between the residents of the Hamilton Beach neighbourhood and the City of Hamilton and the Region of Hamilton-Wentworth.

In 1981 the decision was made to twin the existing Skyway Bridge to handle the ever-increasing traffic flow. As part of the twinning, the QEW was also widened to accommodate the increased traffic. The widening required additional land within the Beach Neighbourhood area, which was acquired by the Ministry of Transportation. By 1988 the twinned bridge was renamed the Burlington Bay-James N. Allen Skyway Bridge and was opened to traffic.

In 1987, the City approved the Hamilton Beach Concept Plan, which was based on the concept of a low-density neighbourhood. In 1992, the Hamilton Beach Neighbourhood Plan was completed and approved. This plan provided details on the location and extent of particular land uses. By-law number 98-281 was passed on November 10,1998, which put restrictions on future development on the west (bay) side of Beach Boulevard. In 1999, the City was in a position to sell the lots they received.


REGION OF HAMILTON-WENTWORTH
Scale 1:175 000 Échelle
1 cm = 1.75 km
10 5 0 5 10 Kilometres
Kilomètres
LAKE ONTARIO
SITE
OAKVILLE
BURLINGTON
Burlington Beach
Hamilton Harbour
DUNDAS
HAMILTON
STONEY CREEK
GLANBROOK
WEST LINCOLN
Mount Nemo
Nelson
Highview
Survey
Waterdown
Hannon
Elfrida
Fulton
Vinemount
Tapleytown
Tweedside
Woodburn
Binbrook
Mt. Hope
Glanford Station
Carluke
Southcote
N. Glanford
Dickenson
Abingdon
Fruitland
Winona
Barton
Queenston
Confederation Park
Hamilton Beach
Burlington St.
Cumberland Ave.
Aberdeen Ave.
King
Main
Stone
Church
Rymal
Twenty
Golf Club
Rd.Club
Airport
Cemetery Rd.
Concession
Mountain
Powerline Rd
Mile
Creek
Ridge
Guyatt
Salem
Chippewa
Kirk
Bell
Lawrence
Limeridge
Mohawk
Upper
Wellington
James
Queen
Dundurn Castle
Royal Botanical Gardens
Hopkins Court
Chedoke G.C.
Chedoke Sanatorium
Hamilton Psych. Hosp.
Hamilton Golf & Country
Glendale G.C.
Falls Cons. Area
Mud
Binbrook Golf Course
Canada Centre for Inland Waters
Burlington G.C.
Hamilton Golf & Country
Fairfield Bldg.
Pier 4 Park
Bayfront Park
Waterfront Trail
Lakeshore
Fairchild
Fairview
Trinity
Fletcher
Nebo
Glancaster
Ferris
Tiddale
Harrison
Woodburn
Burns
Johnston
Bismarck
Green
Smith
Southcote
Kirby Murray Dr.
Sherman Access
Sherman Ave.
Gage Ave.
Ivor Wynne Stad.
Hamilton Golf & Country
Golf Course
Golf Club
Rd.Club
Cemetery Rd.
Airport
Mt. Hope
Glanford Station
Carluke
Binbrook
Woodburn
Fulton
Tapleytown
Tweedside
Vinemount
Winona
Fruitland

1.2 Objectives of Master Drainage Plan

To ensure that the proposed re-development of the Hamilton Beach Neighbourhood proceeds in a manner such that impacts to existing development are mitigated and existing flooding/drainage is improved, the City of Hamilton requires the preparation of a Master Drainage Plan for the area. The Master Drainage Plan will address the unique hydrological characteristics of the development lots west of Beach Boulevard. The main issues that the City of Hamilton requires to be addressed by the Master Drainage Plan are:

- Street flooding – under existing conditions the streets flood periodically, especially in the spring. During these periods of flooding, City staff are required to be on-site to pump the ponding areas to ensure existing development is not impacted and access and egress is possible.
- Individual lot grading – re-development of lots must meet the minimum ground floor elevation of 76.0 m and must not impact existing development due to changes in grade and drainage patterns.

1.3 Background Information

The City of Hamilton provided the following background information:

- Topographic information in digital format (includes storm sewer, property boundaries and topographic contours at a 0.5 m interval)
- Existing plan profiles of the local roads, including Beach Boulevard
- *Hamilton Beach Flood Study* (Dillon, 1987)
- Zoning By-law No. 98-281
- Queen Elizabeth Way widening construction drawings (MTO, CONT No. 86-74)
- Proposal for a *Beach Boulevard Storm Pumping Station*, memo from the Roads Division, City of Hamilton
- *Subsurface Conditions Proposed Eastport Watermain at Beach Boulevard and Eastport Arterial Road, City of Hamilton* (Morton and Partners Limited, 1986)
- Construction Drawings for Eastport Industrial Park Sheets 6A and 7A (Regional Municipality of Hamilton, 1985)
- Construction Drawings for Windermere Basin Rehabilitation, Sheets 9 and 10 (Regional Municipality of Hamilton, 1988)

The information was reviewed and used to complete the tasks of this project. In addition to the above a site visit was conducted on April 6, 1999 to determine flow directions and destinations.

1.4 Approach

1.4.1 Hydrologic Model Development

The OTTSWMM hydrologic model was used to simulate the hydrologic characteristics of the site. OTTSWMM has been used throughout Canada for many years to estimate runoff from all types of land use. The existing and future land uses were taken from the approved land use plan for the Hamilton Beach Neighbourhood (City of Hamilton, 1992).

For the existing development scenario, it was assumed that all lots were developed, except those that are designated for public sale. The properties designated for public sale were identified from the Property Ownership Plans, Part A, B and C (City of Hamilton, 1997).

For the future development scenario, it was assumed that all lots were developed as indicated by the Hamilton Beach Neighbourhood land use plan.

Topographic mapping was obtained from the Region of Hamilton-Wentworth. The available mapping included property boundaries, storm sewer location and size and topographic contours at an interval of 0.5 m. This mapping was used to delineate sub-basin boundaries.

1.4.2 Hydraulic Model Development

The EXTRAN hydraulic model was used to simulate the hydraulic characteristics of the various storm sewers in the study area. EXTRAN has been used extensively throughout Canada to simulate the response of the storm sewer system to an inflow hydrograph.

A majority of the storm sewer information within the Hamilton Beach Neighbourhood, such as pipe inverts and sizes was obtained from the design drawings for the Sanitary Sewer and Forcemain (Regional Municipality of Hamilton-Wentworth, 1992 and 1994).

In addition, the Ministry of Transportation supplied design drawings for the widening of the Queen Elizabeth Way (CONT No. 86-74, WP No. 83-74-05). The design drawings indicated the location of new catchbasins and storm sewers along the length of the Hamilton Beach Neighbourhood as well as any existing catchbasins and storm sewers.

A site visit on April 6, 1999 provided additional information with regards to the location and number of catchbasins and storm sewer outlets along the length of the QEW berm. During the site visit several short duration, high intensity storms moved through the area. This provided an opportunity to observe some of the flooding problems in the previously identified problem areas.

1.4.3 Development of Stormwater Management Alternatives

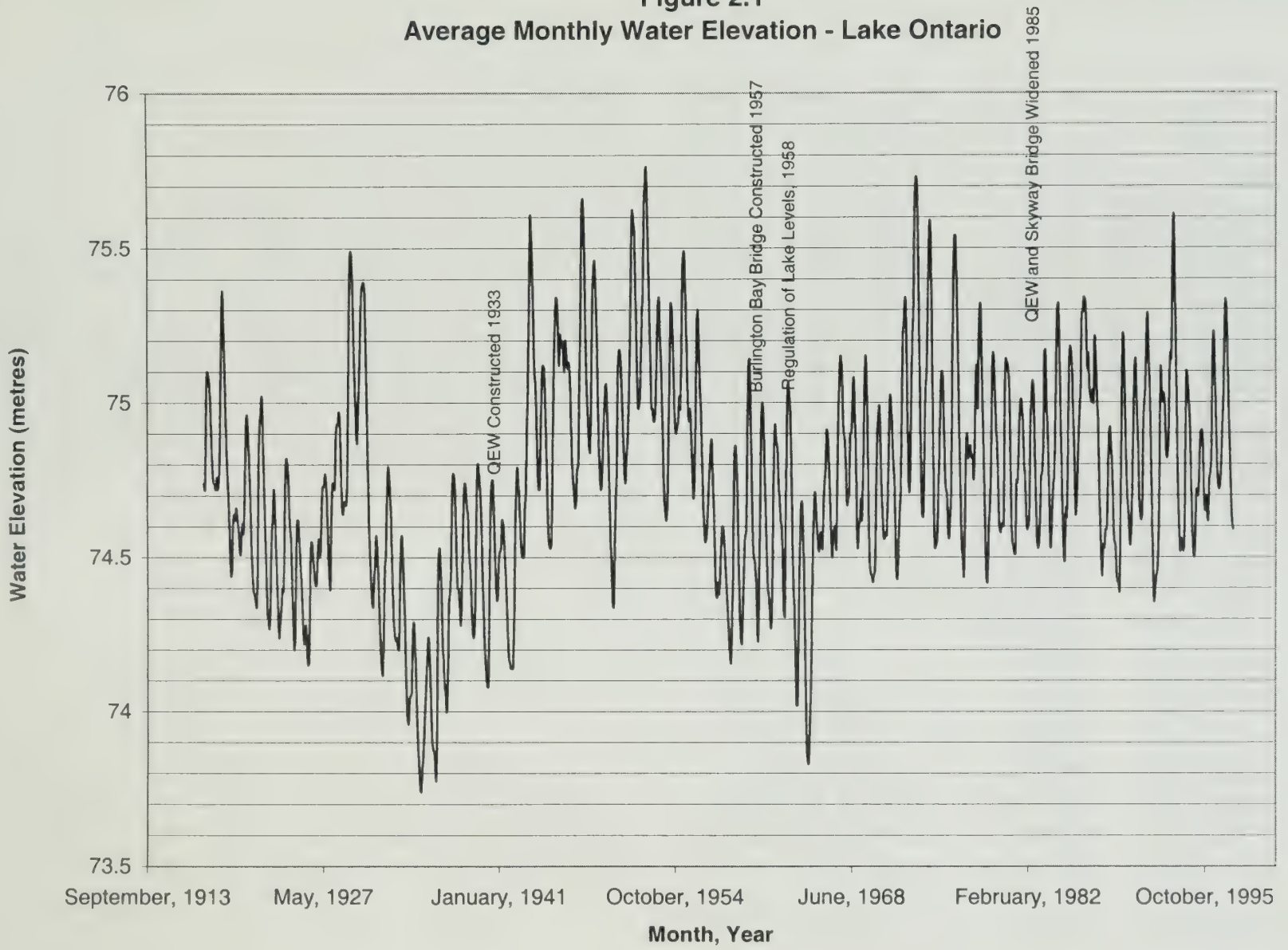
Stormwater management alternatives have been proposed in order to:

- Provide long term alternatives to eliminate/reduce the water on local streets; and

- Mitigate the effects of runoff from new developments on the adjacent properties and the street system.

Each alternative has been investigated to ensure that it is feasible and meets the objectives of the study.

Figure 2.1
Average Monthly Water Elevation - Lake Ontario



road running perpendicular to Beach Boulevard is met. Stormwater then flows towards the QEW. In general there are no catchbasins at the end of these minor roads. In some cases a ditch inlet catchbasin is located in the QEW right-of-way, which outlets to a storm sewer that crosses the highway. In most cases a 300-mm CSP (corrugated steel pipe) conveys overland flows from the street ends to a storm sewer that crosses the highway. For overland flow to reach the QEW right-of-way the water must flow through channeled ditches at the ends of the streets. In general, earthen or wooden berms have been built up at the ends of the streets directly behind the guard rail fence. Overland flow must build up to flow through these channels to reach the ditch inlet catchbasin or culvert.

In some instances there is no overland flow route for stormwater. It is suspected that water will accumulate in low areas and infiltrate into the sandy soils of the beach strip. During high lake levels the capacity of the soils for infiltration will be reduced and a lengthier flooding occurrence would be expected.

2.4 Recurrent Flooding

Nearly every year pumping of stormwater is required to address property owners concerns regarding safety and property damage. Flooding is particularly chronic at Windemere Avenue, Knapmans Drive, Wickham Avenue, Grafton Avenue, Comet Avenue, Granville Avenue, Clare Avenue, Lagoon Avenue and Arden Avenue. This area is lower in elevation than other areas along the beach strip. Flooding has been noted to occur more frequently and severely when Lake Ontario levels are greater than 75.2 m in elevation.

3 HYDROTECHNICAL MODELING

3.1 Hydrologic Modeling

As previously discussed the OTTSWMM hydrologic model was used to simulate the response of the Hamilton Beach Neighbourhood to rainfall for existing and future development scenarios.

3.1.1 Land Use

A majority of the site is residential development. There are pockets of low density attached housing, apartments, commercial and industrial land uses. Figures 3.1 and 3.2 illustrate the existing land uses and their location.

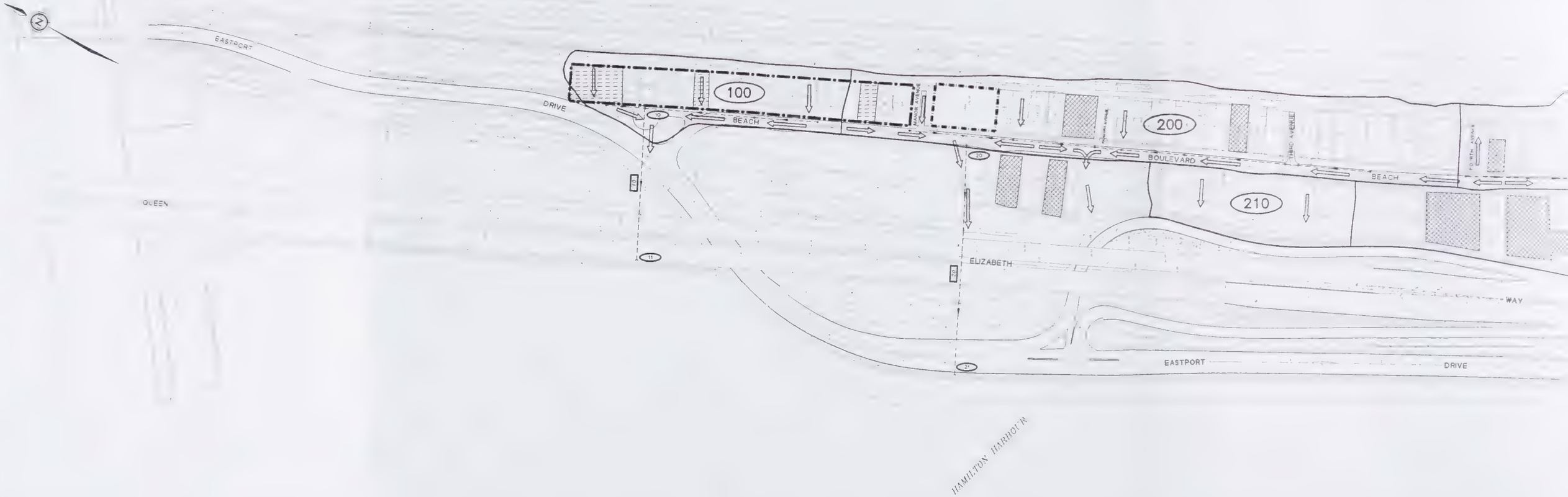
Table 3.1 provides an estimate of the percent land use in the Hamilton Beach Neighbourhood for existing and future development scenarios. For hydrologic modeling purposes a percent imperviousness was assumed for each type of land use. Table 3.1 also provides a tabulation of the imperviousness assumed for each land use type.

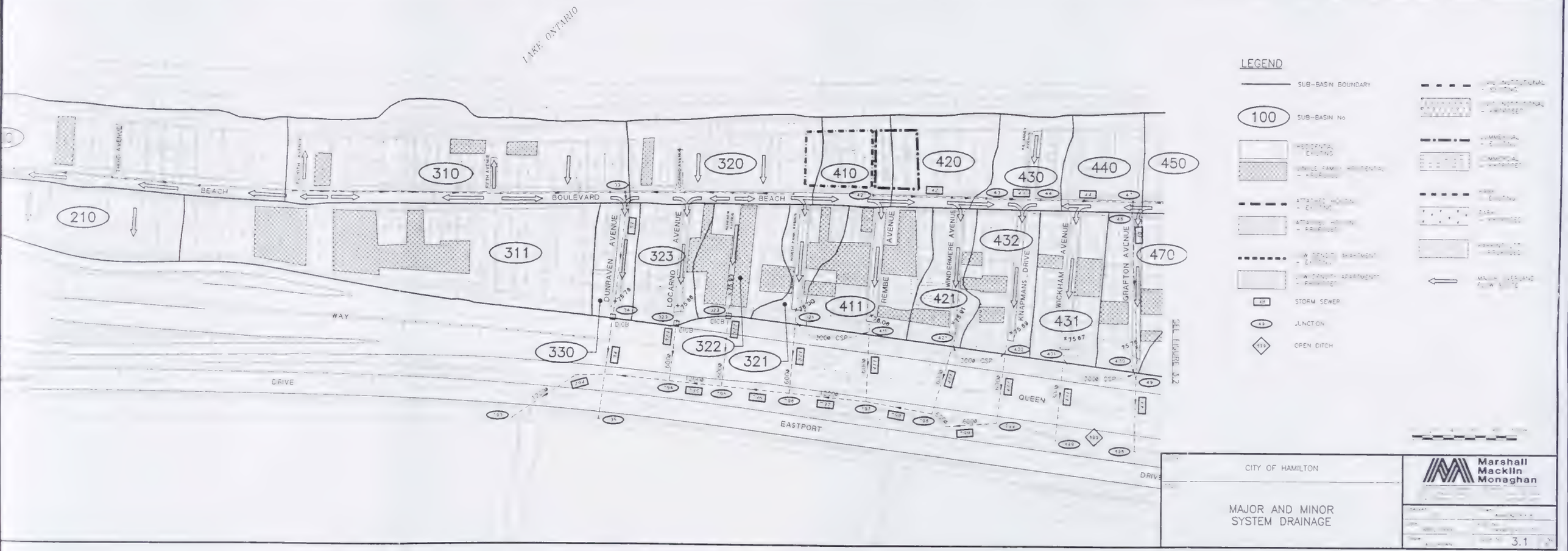
TABLE 3.1
Percent Land Use Existing and Future Development

Land Use Description	Percent of Hamilton Beach		Percent Imperviousness
	Existing	Future	
Grassed Buffer	13.4	13.4	0
Road right-of-way	12.4	12.4	85
Attached Housing	3.3	5.3	60
Commercial	4.3	5.4	75
Park	3.7	4.1	0
Civic-Institutional	3.3	3.7	90
Low Density Apartment	1.0	1.3	65
Single Family Residential	46.3	53.3	35
Undeveloped	12.2	0.0	0
Parking Lot	0.0	1.0	90

3.1.2 Sub-basin Delineation

The Beach Neighborhood was delineated into sub-basins. This delineation was based on topographic features, storm sewer and catchbasin locations. Figures 3.1 and 3.2 illustrate the sub-basin numbers and boundaries.


EASTPORT
DRIVE
100
BEACH
200
BOULEVARD
BEACH
210
QUEEN
ELIZABETH
WAY
EASTPORT
DRIVE
HAMILTON HARBOUR



CITY OF HAMILTON

MAJOR AND MINOR SYSTEM DRAINAGE

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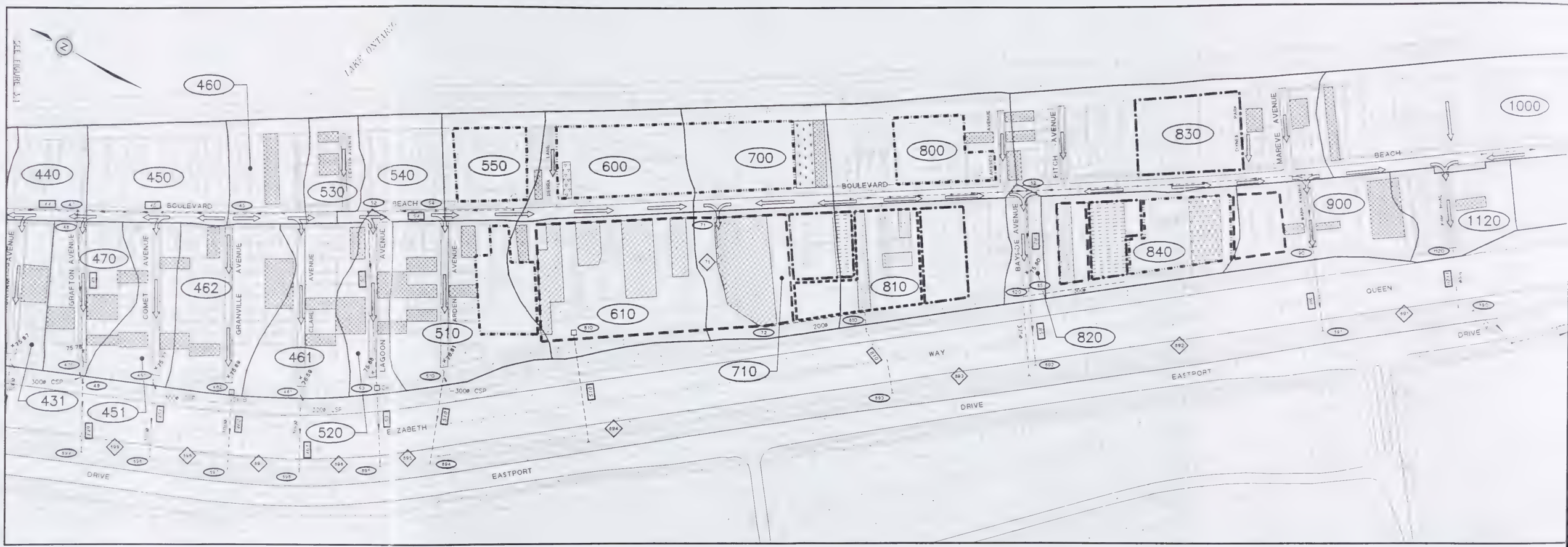
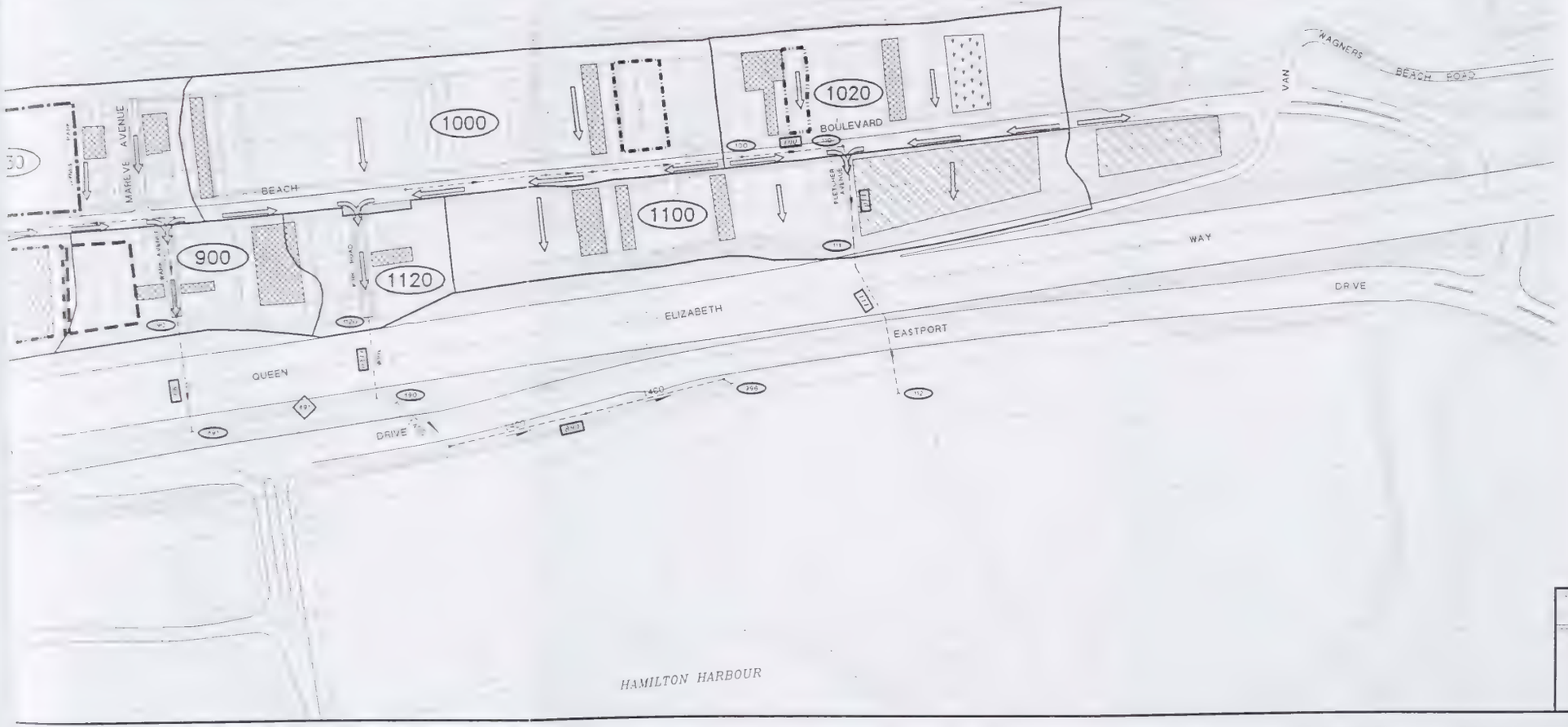
3.1


LAKE ONTARIO
460
440
450
530
540
550
600
700
800
830
1000
BOULEVARD
BEACH
900
1120
840
470
462
461
510
610
810
820
710
431
451
520
GRAFTON AVENUE
COMET AVENUE
GRANVILLE AVENUE
CLARE AVENUE
LAGOON AVENUE
ARDEN AVENUE
BAYSIDE AVENUE
FITCH AVENUE
MAREVE AVENUE
QUEEN
WAY
DRIVE
EASTPORT
ELIZABETH



LEGEND

- 440
- RESIDENTIAL – EXISTING
- SINGLE FAMILY RESIDENTIAL – PROPOSED
- ATTACHED HOUSING – EXISTING
- ATTACHED HOUSING – PROPOSED
- LOW DENSITY APARTMENTS – EXISTING
- LOW DENSITY APARTMENTS – PROPOSED
- STORM SEWER
- JUNCTION
- OPEN DITCH

CITY OF HAMILTON

MAJOR AND MINOR SYSTEM DRAINAGE

Marshall Macklin Monaghan

3.2

TABLE 3.2 Sub-basin Parameters

Sub-basin #	Area (ha)	Length of Street Seg. (m)	# of Inlets to Storm Sewer	Inlet Type	Width of Sub-basin (m)	Percent Impervious	
						Existing	Future
100	1.80	200	5	A	45	49.9	59.6
200	4.57	480	14	A	60	38.7	39.8
210	1.29	60	0	-	60	29.0	29.0
310	2.81	320	9	A	60	34.4	36.0
311	3.27	90	0	-	90	21.6	30.2
320	1.54	180	8	A	60	34.1	35.2
321	0.49	100	1	C	30	33.9	41.4
322	0.86	100	1	C	95	18.7	35.0
323	0.72	100	1	C	40	36.7	39.3
330	0.45	100	1	C	45	45.0	45.0
410	0.60	60	2	A	75	29.3	29.3
411	1.16	120	1	C	90	25.8	37.2
420	0.86	80	2	A	65	41.7	41.7
421	0.62	120	1	C	25	36.1	39.7
430	0.65	85	5	A	65	46.4	46.4
431	0.92	120	1	C	65	35.6	40.1
432	0.86	120	1	C	65	35.7	40.1
440	0.70	65	2	A	85	40.5	40.5
450	1.40	140	4	A	80	37.4	37.4
451	0.91	150	1	C	40	37.3	41.5
460	0.82	70	4	A	85	31.4	35.8
461	1.33	150	1	C	70	30.0	33.1
462	1.64	150	1	B	80	34.1	39.1
470	1.08	150	1	C	55	34.9	40.6
510	2.08	140	1	C	120	22.5	26.9
520	0.77	165	4	A	20	35.3	43.2
530	0.55	35	2	A	50	34.6	38.7
540	0.91	80	3	A	85	38.5	38.5
550	0.99	70	2	A	85	16.9	17.0
600	1.58	170	4	A	85	67.6	69.7
610	2.35	100	0	-	85	33.6	50.5
700	1.46	135	1	A	80	61.7	63.5
710	1.75	100	0	-	120	35.7	54.2
800	1.88	180	9	A	80	28.4	29.6
810	1.93	100	1	A	100	42.5	53.6
820	0.40	80	1	B	35	44.6	44.6
830	3.21	280	11	A	85	50.0	51.7
840	1.77	90	1	C	85	36.9	63.2
900	1.36	75	2	A	75	31.8	36.9
1000	3.90	375	6	A	80	36.2	37.4
1020	2.29	250	3	A	70	38.2	40.9
1100	3.12	75	1	C	60	19.5	40.0
1120	0.88	80	1	C	90	32.9	35.0

Table 3.2 provides a tabulation of the sub-basin parameters input into the hydrologic and hydraulic models. The inlet types presented in Table 3.2 have been arbitrarily assigned a letter, A, B or C. Inlet Type A includes the standard road catchbasins. The catchbasin covers may vary within a street segment between a grid or herringbone pattern. Inlet Type B includes the ditch-inlet catchbasins at the end of the side streets that end at the Queen Elizabeth Way noise berm. Inlet Type C includes the 300-mm diameter corrugated steel pipes at the end of the side streets that end at the Queen Elizabeth Way noise berm.

3.1.3 Rainfall Characteristics

Rainfall Data from the Hamilton Royal Botanical Garden was used to generate the design rainfall events for the simulations. A six-hour Chicago storm with a ten-minute time step was selected for the simulations. This storm was considered suitable since it provides a balance between rainfall volume and peak intensity. Table 3.3 provides a summary of the rainfall parameters.

TABLE 3.3
Rainfall Characteristics – 6 Hour Chicago Distribution
Hamilton RBG Rainfall Gauge

Return Period Storm	Rainfall Depth (mm)	Peak Rainfall Intensity (mm/hr)
2-year	35.2	69.7
5-year	48.7	90.4
10-year	57.7	103.3
25-year	69.1	120.8
50-year	77.5	132.8
100-year	85.9	145.1

3.1.4 Soil Infiltration Characteristics

A geotechnical investigation of the site was completed as part of the proposal for the installation of the Eastport watermain. All boreholes indicated that the soils are dark brown sands with pockets of gravel mixed in. The boreholes were drilled to a depth of at least three metres.

The OTTSWMM hydrologic model uses the Horton equation to determine how much of the rainfall will infiltrate into the soil. Three parameters are required to apply this equation, the minimum infiltration rate (mm/hr), the maximum infiltration rate (mm/hr) and the decay parameter (sec^{-1}). For sandy soils the minimum infiltration rate was estimated to be 25 mm/hr; the maximum infiltration rate was estimated to be 125 mm/hr and the decay rate was estimated to be 0.00056 sec^{-1}.

OTTSWMM does not directly have the capability of simulating situations where the water table elevation varies.

3.2 Hydraulic Modeling

As previously discussed the EXTRAN hydraulic model was used to simulate the response of the Hamilton Beach storm sewer systems to the volume of runoff from the site for existing and future development scenarios. Data for the location, size and inverts for the storm sewers was obtained from the topographic mapping provided by the Region of Hamilton-Wentworth and the construction drawings for the sanitary sewer and forcemain for the Hamilton Beach Neighbourhood. Additional information was obtained from the MTO construction drawings for the widening of the QEW.

There are eight main storm sewer systems that collect runoff from Beach Boulevard and lands east of Beach Boulevard (lake side). A small portion of lands west of Beach Boulevard (bay side) also drain to the storm sewer systems. These system outlets are designated with the numbers 10, 20, 34, 49, 53, 85, 90 and 111 (Figures 3.1 and 3.2).

Lands west of Beach Boulevard generally drain to a ditch inlet catchbasin (DICB) or a 300-mm diameter CSP culvert, which convey the overland flow to Hamilton Harbour via a storm sewer under the QEW.

The drainage area to each of the storm sewer outlets and its characteristics has been tabulated in Table 3.4. Details regarding the number of catchbasins and their characteristics are provided in Table 3.2 for each of the sub-basins.

TABLE 3.4
Storm Sewer Outlet Characteristics

Storm Sewer Outlet	Drainage Area (ha)	Storm Sewer Diameter (mm)	Estimated Storm Sewer Invert (m)
10	1.80	600	75.04
20	4.57	600	75.02
34	4.80	600	74.55
49	6.11	600	74.36
53	3.23	525	74.37
85	5.08	600	74.63
90	1.36	300	74.79
111	9.32	600	73.95

For simulation purposes the EXTRAN model was run for two different water elevations at the outlet. The 'low' water level was set at 74.6 m, which is an average of the two target extremes (see Section 2.1). The 'high' water level was set at an elevation of 75.4 m, which is the maximum target for the summer. Historically, the 'high' water level has been exceeded on occasion, nevertheless it provides a target to base future recommendations. Hamilton Harbour water levels greater than 75.6 m

will cause flooding of the Hamilton Beach roads along the QEW noise berm from water backing up through the storm sewer systems.

The estimated storm sewer inverts are located just upstream of the QEW noise berm. As is evident from the values provided in the table, it is expected that the storm sewer systems will operate under surcharged or partially surcharged conditions for much of the year.

4 MODELING RESULTS

4.1 *Existing Development – Existing Storm Sewer System*

The existing development model was simulated for two different lake levels. A 'low' lake level of 74.6 m was used as representative of an average lake level. A 'high' water level of 75.4 m was used as representative of an average high summer water level.

The storm sewer along Beach Boulevard and at the side streets will remain unsubmerged with a lake level of 74.6 m. However, the outlets of the storm sewers to Hamilton Harbour will be submerged. The simulations indicate that the storm sewers have the capacity to convey the stormwater runoff from the area without surcharging to the street. The corrugated steel pipes and the ditch inlet catchbasins along the QEW noise berm do not have the capacity to convey stormwater from the existing development without ponding on the streets and yards. This occurs specifically at the base of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden. The simulations indicate that some level of ponding (flooding) will occur as frequently as a 1 in 2-year storm.

With a high lake level of 75.4 m many of the storm sewers along the side streets will be submerged as well as the outlets to Hamilton Harbour. Although the storm sewers are submerged the simulations indicate that most will have the capacity to convey stormwater flows without surcharging onto the streets. As with the low lake level simulations, prolonged ponding (flooding) will occur at the base of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden. The flooding of the streets and yards will occur more frequently since runoff must pond to a higher depth due to the higher lake levels. Flooding events will also be longer in duration.

In general, areas of ponding may have an adverse affect on highway embankment, especially if the ponding level reaches the subgrade (assume one-metre below top of road). In the area of the prolonged ponding an earthen 'noise' berm has been constructed between the Hamilton Beach Neighbourhood and the Queen Elizabeth Way. Therefore, the ponding is not against the highway embankment and will not impact the subgrade material.

4.2 *Future Development – Existing Storm Sewer System*

The same scenarios as above were simulated with the future redevelopment of vacant lots. This redevelopment will increase the volume of runoff due to the higher imperviousness of the area. The simulations indicated that the storm sewer would have the capacity to convey stormwater runoff from the site for lake levels as high as 75.4 m.

Similar results as above were noted from the simulations in regards to the frequency of flooding at the base of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden. Due to the increased volume of runoff the duration of flooding events would be increased.

4.3 Options to Alleviate Flooding

Redevelopment of Hamilton Beach neighbourhood will increase the duration of flooding events at the base of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden. Therefore, several options were considered in order to alleviate the flooding in this area.

Based on a preliminary analysis it appears that the flooding is not necessarily the result of water backing up the storm sewer from Hamilton Harbour. Flooding occurs more frequently when lake levels are high partly because stormwater must pond to a higher elevation to pass through the submerged storm sewers. In some cases existing culverts under the QEW noise berm are set higher than the end of the street, therefore water must pond before it has an outlet no matter what the outlet conditions are.

The OTTSWMM and EXTRAN models (future proposed development conditions) were modified for each of the options to determine what affect each of the options would have on the existing street flooding. For all the options a lake level of 75.4 m was used. For illustration purposes the simulated 1 in 10 year flood level has been sketched on a plan in the area of concern.

Option 1 (Continue Current Practices): This option includes non-construction means to improve drainage to the outlets under the QEW berm. It is recommended to remove any earthen or wooden berms at the street ends, which obstruct the flow of water to the QEW ditch and to ensure outlets are free of sediment, vegetation and garbage. Figure 4.1 illustrates the simulated 1 in 10 year storm flood level (lake level 75.4 m) for the existing storm sewer and future proposed development.

Option 2 (Gravity Drainage Improvements): Convey stormwater by gravity to a new outlet under the QEW. Provide ditch inlet catchbasins at the end of Wickham, Grafton, Comet, Granville, Clare and Arden and collect stormwater in a pipe running parallel to the QEW noise berm. A new outlet pipe at the end of Granville Avenue will convey stormwater across the QEW to the ditch between Eastport Drive and the QEW. The pipe will be large enough to ensure peak flows will be conveyed under low head conditions during high lake levels.

Figure 4.2 illustrates the simulated 1 in 10 year storm flood level (lake level 75.4 m) for Option 2. The simulations indicate that there would be some flooding at the ends of the streets. The simulations indicate that water levels in the ditch between the QEW and Eastport Drive will not increase in elevation due to the increased flows from the Hamilton Beach Neighbourhood. This may be explained due to the fact that the system operates passively with the difference in water surface elevations between Lake Ontario, the ditch and the Hamilton Beach Neighbourhood driving the flows. The ditch has a very large capacity available to store water relative to the volume of water from the Hamilton Beach Neighbourhood. Therefore, the increase in flow to the ditch should not result in an increased depth of water.

Option 2a (Gravity Drainage Improvements with Larger Storm Sewers): Similar to Option 2 but storm sewer sizes were increased to the next larger pipe diameter. Simulations indicate that the larger storm sewers would pass the 1 in 10 year storm without surcharging to the street surface.

Figure 4.2a illustrates the simulated 1 in 10 year storm flood level (lake level 75.4 m) for Option 2a. The simulations indicate that there would be no flooding at the ends of the streets. As discussed in Option 2, the simulations indicate that water levels in the ditch between the QEW and Eastport Drive will not increase in elevation due to the increased flows from the Hamilton Beach Neighbourhood.

Option 3 (Pumping to Existing Sewer): Convey stormwater to an outlet by pumping to an existing storm sewer. Provide catchbasins at the end of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden and collect stormwater in a pipe running parallel to the QEW noise berm. A lift station at Knapmans Drive would pump stormwater to an existing storm sewer that runs between Eastport Drive and the QEW. Different pump rates were reviewed to determine the impact pumping will have on the downstream storm sewer. This option may include a new pipe under the QEW.

Figure 4.3 illustrates the simulated 1 in 10 year storm flood level (lake level 75.4 m) for Option 3. Since the existing sewer has a limited capacity to accept additional flows the pump rate is also limited. Even though the simulated flood levels are similar to that under current conditions the duration of flooding would be shorter.

Option 4 (Pumping to Existing Ditch): Convey stormwater to an outlet by pumping to an existing ditch between Eastport Drive and the QEW. Similar to Option 3 except the lift station will be located at the end of Granville Avenue and water will be pumped to the existing ditch. Different pump rates were reviewed to determine the impact pumping will have on the downstream ditch. This option includes a new forcemain under the QEW.

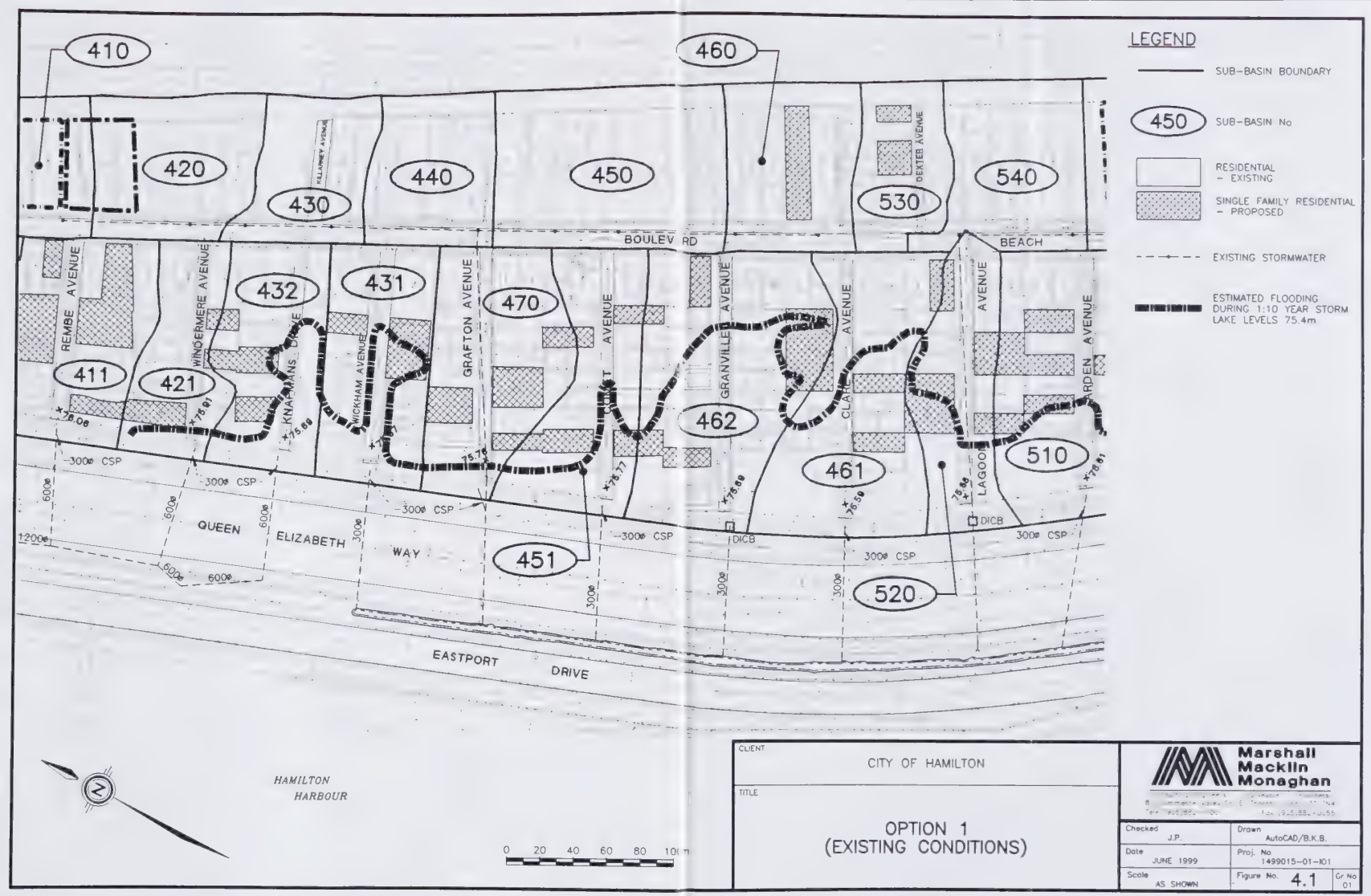
Figure 4.4 illustrates that there would be no flooding of the area during a 1 in 10-year storm (lake level 75.4 m) for Option 4. In fact the simulations indicate that a maximum pump rate of 0.6 m^3/s would reduce flooding during a 1 in 100-year storm and a lake level of 75.4 m.

Option 5 (Lower Tailwater by Pumping Existing Ditch): Water levels in the ditch located between the QEW and Eastport Drive would be kept low by installing a pump lift station at the downstream end of the ditch (opposite Kirk Road). By lowering the water level in the ditch, storm sewer outlets from the Hamilton Beach Neighbourhood would not be submerged.

Figure 4.5 illustrates the simulated 1 in 10 year storm flood level (lake level 75.4 m) for Option 5. The simulations indicate that a portion of the area would benefit from pumping water from the ditch (specifically Knapman's and Wickham), however, flood levels would not change for the rest of the area. The duration of flooding would be reduced compared to current practices.

4.3.1 Evaluation of Options

All the options were then evaluated technically and economically. The following sections describe how this evaluation was carried out and provide a summary of the evaluation.



CLIENT: CITY OF HAMILTON

TITLE: OPTION 1 (EXISTING CONDITIONS)

Marshall Macklin Monaghan

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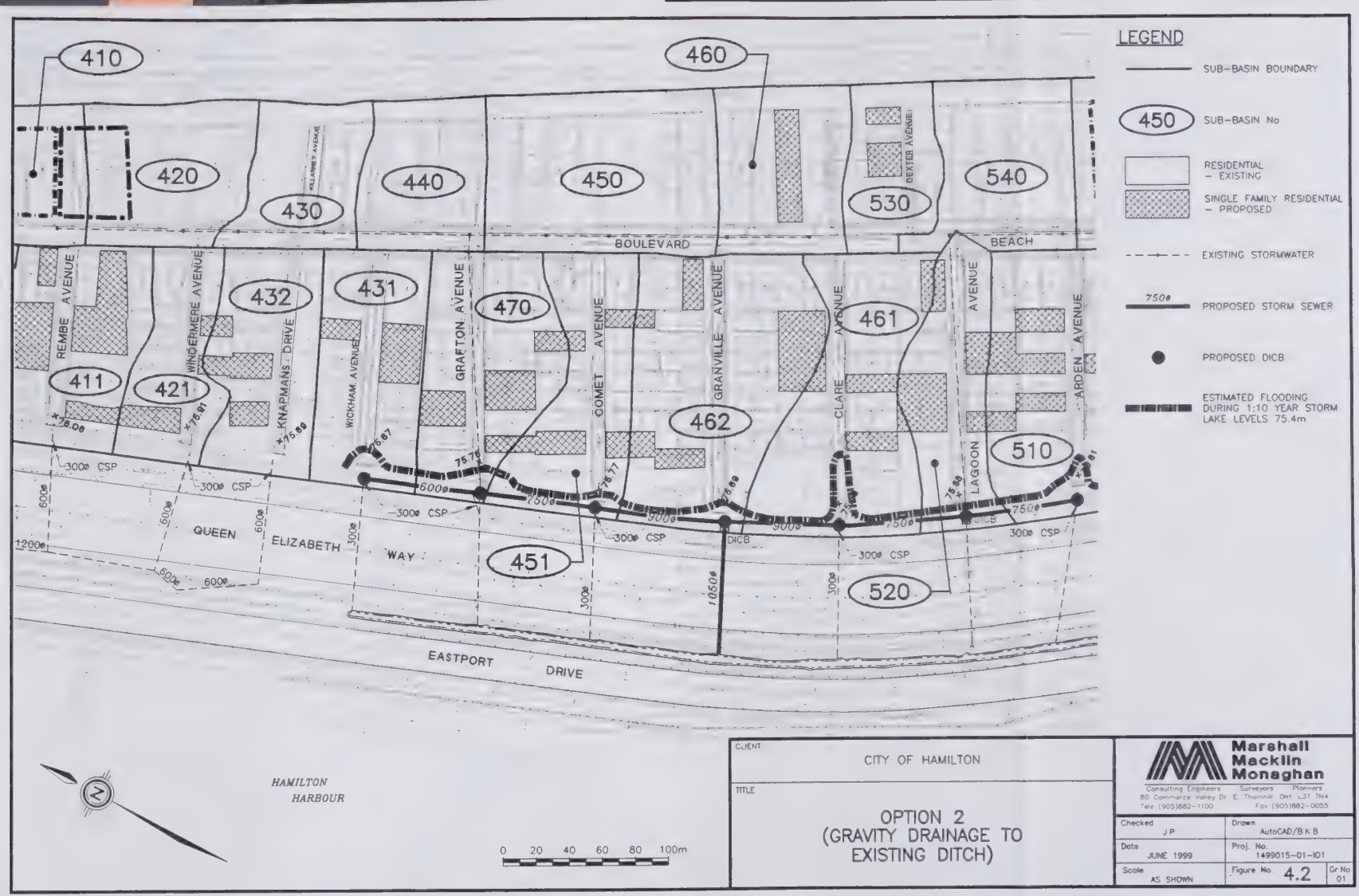
Date: JUNE 1999	Proj. No: 1499015-01-101
Scale: AS SHOWN	Figure No: 4.1 Gr No: 01



CLIENT: CITY OF HAMILTON

TITLE: OPTION 2 (GRAVITY DRAINAGE TO EXISTING DITCH)

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Date: JUNE 1999	Proj. No.: 1499015-01-101
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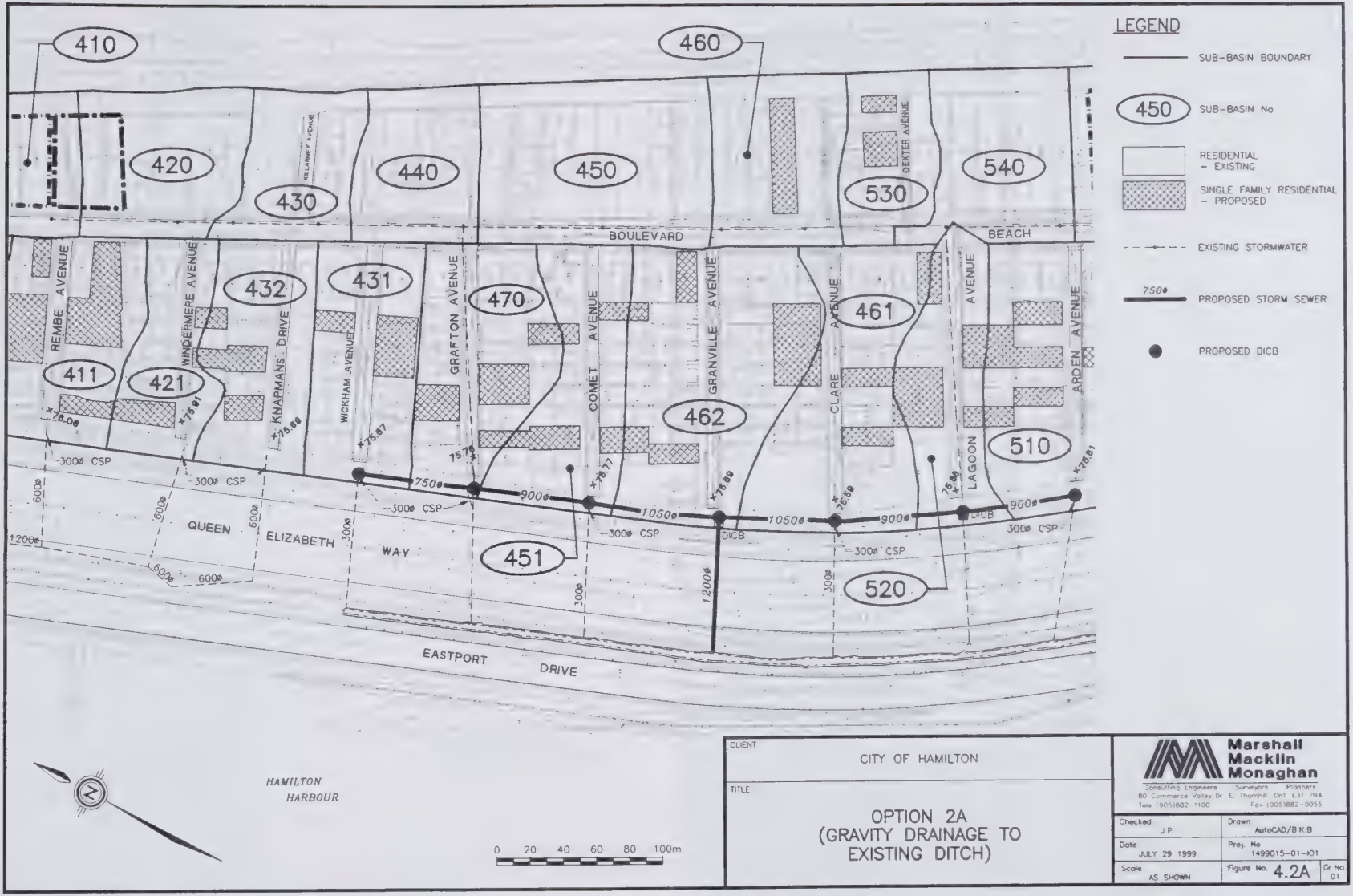

CLIENT
CITY OF HAMILTON

TITLE
OPTION 2A
(GRAVITY DRAINAGE TO EXISTING DITCH)

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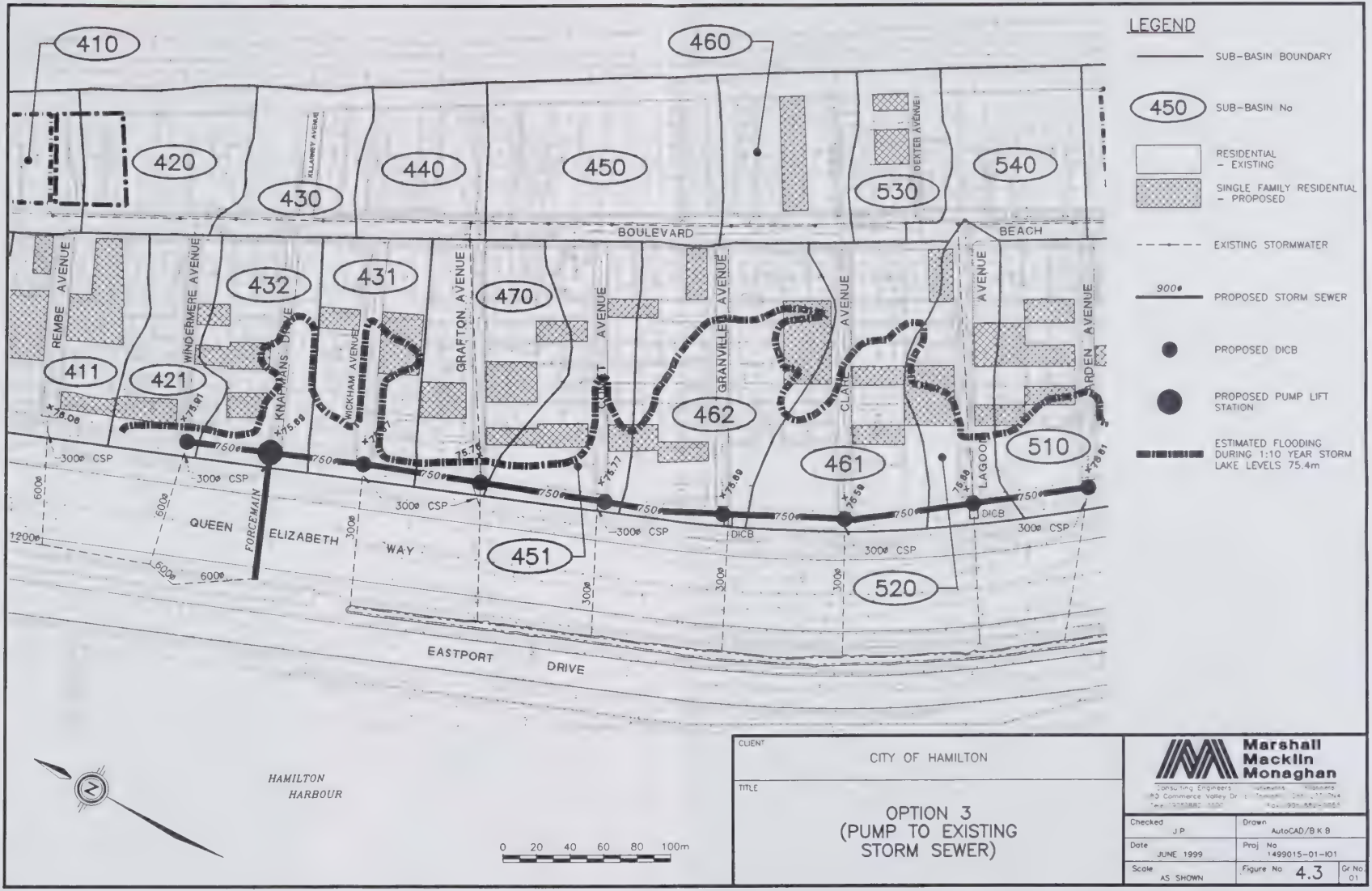
Checked J.P	Drawn AutoCAD/B K.B
Date JULY 29 1999	Proj. No 1499015-01-I01
Scale AS SHOWN	Figure No. 4.2A Gr No 01



CLIENT: CITY OF HAMILTON

TITLE: OPTION 3 (PUMP TO EXISTING STORM SEWER)

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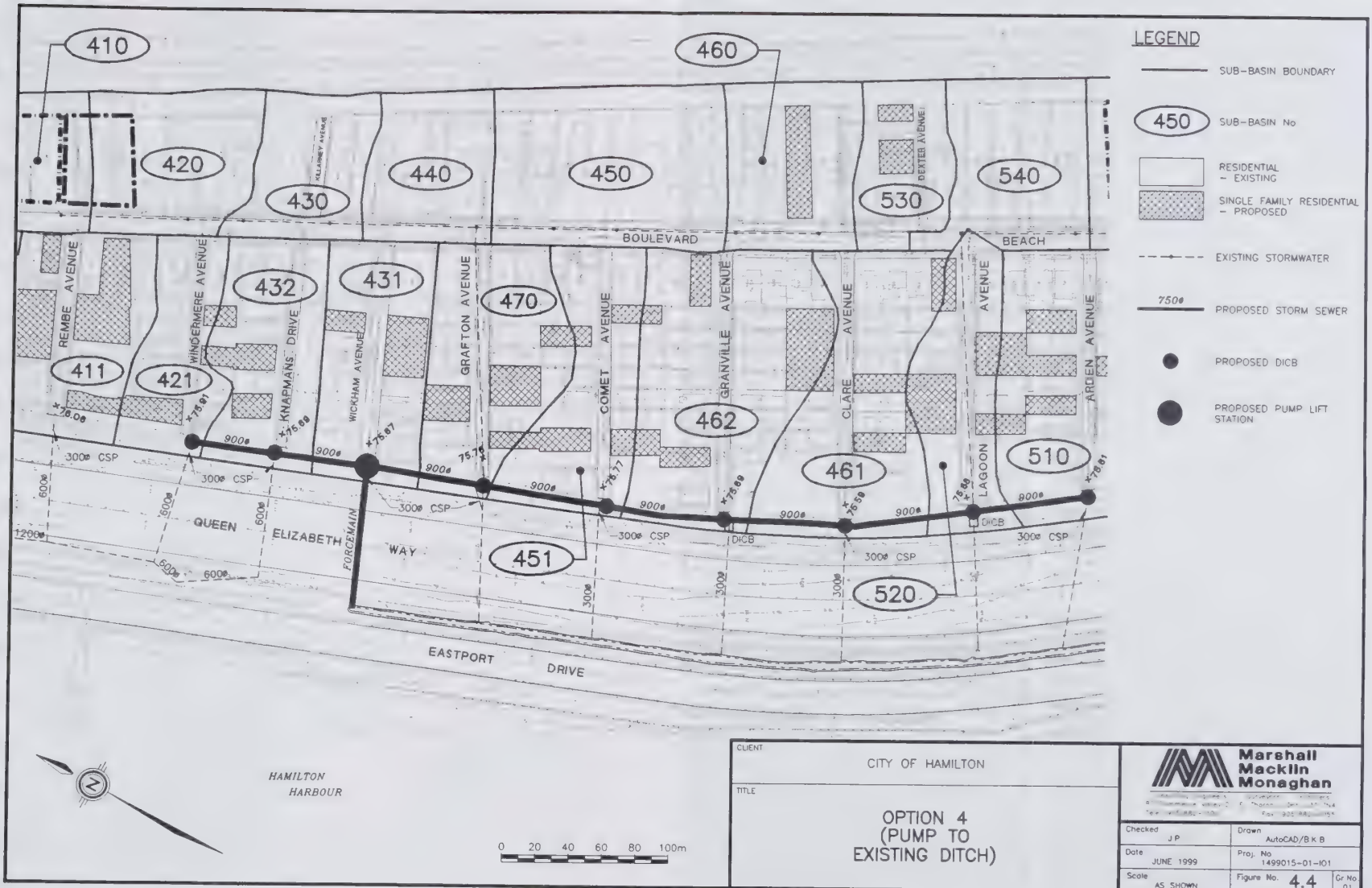
Date: JUNE 1999	Proj. No: 14990I5-01-101
Scale: AS SHOWN	Figure No: 4.3 Gr No: 01



CLIENT: CITY OF HAMILTON

TITLE: OPTION 4 (PUMP TO EXISTING DITCH)

Marshall Macklin Monaghan

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Date: JUNE 1999	Proj. No: 1499015-01-101
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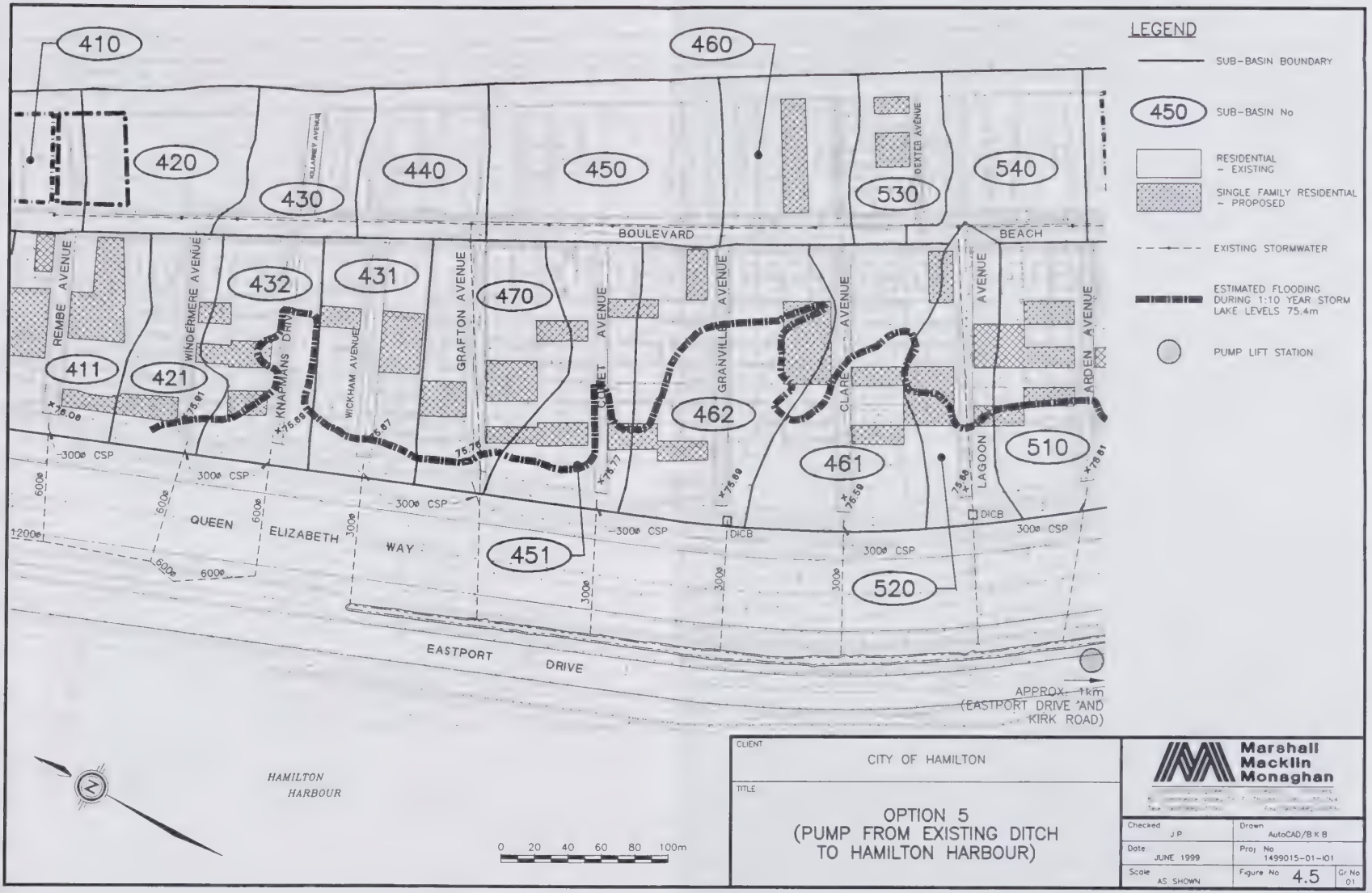

CLIENT: CITY OF HAMILTON

TITLE: OPTION 5 (PUMP FROM EXISTING DITCH TO HAMILTON HARBOUR)

Marshall Macklin Monaghan

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Date JUNE 1999	Proj. No. 1499015-01-101
Scale AS SHOWN	Figure No. 4.5 Gr. No. 01

4.3.1.1 Technical Evaluation

The technical evaluation was based on the level of protection that would be provided by each proposed option and it's ability to meet the goal of the study.

> *"[Provide] long term alternatives to eliminate/reduce the water on local streets."*

For the computer simulations a starting lake level of 75.4 m was assumed for the analysis. The computer models simulated a range of design rainstorms (2-year, 5-year, 10-year, 25-year, 50-year and 100-year). Since lake levels will vary from year to year, month to month and day to day, it is necessary to determine the probability of a design storm (2-year to the 100-year) occurring when the water elevation in the lake is at 75.4 m.

Combined Statistical Analysis of Storm Events and Lake Levels

The HYDSTAT computer program (developed by the Ontario Ministry of Natural Resources in 1982) was used to determine the combined probability of the occurrence of storm events and water levels in Lake Ontario.

The recurrence interval of lake levels for Lake Ontario at Burlington were extracted from a report (*Great Lakes System Flood Levels and Water Related Hazards,* 1989) published by the Conservation Authorities and the Ministry of Natural Resources. This report analyzed the average monthly lake levels and the instantaneous peak water levels that have been recorded over an 87-year period (1900 to 1987). The statistical analysis for the Hamilton Beach Master Drainage Plan has been based on the instantaneous peak water levels since that would provide the most conservative analysis. The instantaneous peak lake elevations for Lake Ontario at Burlington have been extracted from the report and are tabulated in Table 4.1 (column 2).

The lake level elevations are based on the International Great Lakes Datum (IGLD) 1955. Topographic mapping for the Hamilton Beach area is based on the Canadian Geodetic Datum (CGD). The elevation difference between the IGLD (1955) and the Geodetic Datum (1985) is negligible (personal communications with Ralph Moulton, Manager of Great Lakes-St. Lawrence Water Level Information Office in the Hamilton area).

In order to statistically combine the recurrence interval of lake levels and storm events it is necessary to estimate the depth of water that would result during each storm event independent of the lake level. This was accomplished by estimating the design storm peak flow from a sub-basin. For the purposes of this report sub-basin 470 was used as representative of the area. The depth of ponding that would be required to convey this flow through the available outlet was then simulated (using a low lake level) and have been tabulated in Table 4.1 (column 3). This estimate takes into consideration the attenuation of the peak flows due to the storage available for ponding at each of the outlets.

Both frequency curves were then input into the HYDSTAT program. The resultant combined frequency of flood elevations (lake elevation plus ponding depth) was estimated to be as follows. Details of the analysis are provided in Appendix B.

TABLE 4.1
Combined Frequency Analysis for Lake Levels and Flooding Depths

(1) **Return Period**	(2) **Instantaneous Peak Lake Level**	(3) **Depth of Water**	(4) **Combined Frequency**
2 year	75.28 m	0.18 m	75.44 m
5 year	75.49 m	0.23 m	75.65 m
10 year	75.61 m	0.25 m	75.76 m
25 year	75.75 m	0.28 m	75.89 m
50 year	75.84 m	0.34 m	75.98 m
100 year	75.93 m	0.38 m	76.05 m

Table 4.2 provides a tabulation of the flood levels for the design storms derived from the computer simulations representing the existing storm sewer with future proposed development. As previously discussed the constant lake level used in the model was 75.4 m. The table also provides an estimate of the frequency of flood levels (derived from column 4 of Table 4.1). It is possible to estimate the level of flood protection that would be provided by each of the options considered. For example simulations of Option 2 indicate that flooding would be reduced during a 1 in 10-year storm but not during a 1 in 25-year storm (with a constant lake level of 75.4 m). This indicates that the level of protection would be greater than a 1 in 10-year level.

TABLE 4.2
Frequency of Flooding Depths Lake Level 75.4 m

Design Storm Frequency	**Flood Level (m)**	**Estimated Flood Level Frequency (based on column 4 of Table 4.1)**
2	75.66	5 year
5	75.77	10 year
10	75.80	>10 year
25	75.83	<25 year
50	75.85	<25 year
100	75.87	25 year

4.3.1.2 Economic Evaluation

Ten years of data were available, from the City of Hamilton, regarding the annual cost of temporary pumping during spring periods. For comparison purposes these costs were adjusted to 1999 dollars and are shown on Figure 4.7. For illustration purposes the average monthly water levels in Lake Ontario have also been graphed (Figure 4.6). As the graphs illustrate some level of pumping was required for every year when the lake levels exceeded 75 m. For example Lake Ontario water levels remained below 75 m during 1995 and there was no pumping required.

The collected data spans a wide range of flooding conditions on the Beach strip and therefore provides a good indication of pumping costs in the future. The highest average monthly lake level in 1993 was 75.6 m and in 1998 was 75.4 m. A statistical analysis of the average monthly lake levels (*Great Lakes System Flood Levels and Water Related Hazards*, Ministry of Natural Resources, 1989) indicates that the 1993 water level has a return period of greater than 1 in 200 years and the 1998 water level has a return period of approximately 1 in 50 years.

The average annual cost of pumping is estimated to be $60,600 over the ten-year period (1989 to 1998). This value was used in the following analysis to provide a base case for comparison purposes.

Figure 4.6
Average Monthly Lake Levels, Lake Ontario

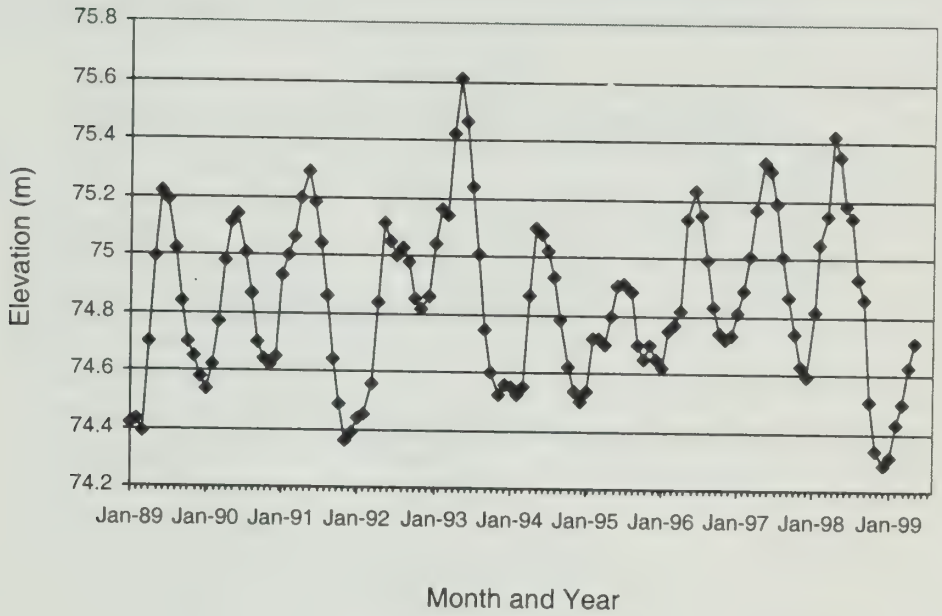
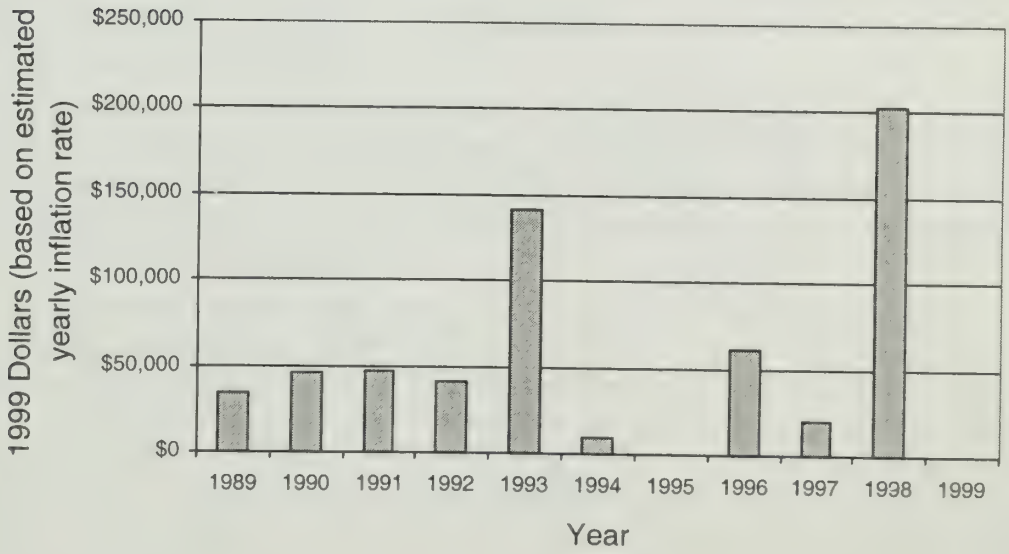


Figure 4.7
Pumping Cost Estimate Hamilton Beach Neighbourhood



Present Value Cost Analysis

The economic analysis of the different options was based on the costs of construction, operation and maintenance and the need for back-up flood protection measures (dependent on the level of protection provided). The costs of materials and construction have been estimated for each of the options considered. These costs are tabulated in Table 4.3 and include a 25% engineering and contingency fee.

TABLE 4.3
Preliminary Cost Estimate for Construction

Option	Item Description	Estimated Cost
2	Storm sewers Ditch Inlet Catchbasins Tunneling under QEW (84 m – 1050 mm)	$520,000
2a	Storm sewers Ditch Inlet Catchbasins Tunneling under QEW (84 m – 1200 mm)	$570,000
3	Storm sewers (545 m – 750 mm) Ditch Inlet Catchbasins Pumping lift station 400 mm forcemain under the QEW	$1,040,000
4	Storm sewers (545 m – 900 mm) Ditch Inlet Catchbasins Pumping lift station 400 mm forcemain under the QEW	$1,050,000
5	Pumping lift station	$200,000

Table 4.4 provides a summary of the assumptions made for the operation and maintenance costs for each of the options.

The maintenance cost for the Options 2 and 2a have been estimated at $500 per year. This will include yearly removal of any garbage and/or vegetation, which may obstruct the ditch inlet catchbasins. As well, periodic removal of sediment from the storm sewer may be required since the grade will be relatively flat and the velocities of flow through the system low.

Operation and maintenance costs for Options 3 and 4 are similar since both rely on a pumping lift station to lower storm water levels in the Hamilton Beach Neighbourhood. It is expected that the pumps will only operate during storm events. Option 5 also relies on a pumping lift station however it is located within the ditch between the QEW and Eastport Drive. The intent of this system will be to keep water levels in the ditch at a low elevation such that the storm sewer outlets from the Hamilton Beach Neighbourhood are not submerged. It is expected that this pump station will operate nearly continuously since seepage of water from Hamilton Harbour will occur. Also, it is necessary to keep the water levels low at all times since the exact timing and intensity of a storm event would be difficult to predict in advance.

TABLE 4.4
Estimation of Operation and Maintenance Costs

Option	Task Description	Estimated Cost	Interval
2	Garbage/sediment removal	$500	1 year
2a	Garbage/sediment removal	$500	1 year
3	Electricity Pump overhaul New pump	$500 $5,000 $43,000	1 year 7 years 20 years
4	Electricity Pump overhaul New pump	$500 $5,000 $43,000	1 year 7 years 20 years
5	Electricity Pump overhaul New pump	$1,200 $5,000 $43,000	1 year 3 years 9 years

Table 4.5 provides a summary of the assumptions made for the present value cost analysis of the average annual costs associated with back-up pumping for the different options.

The average annual cost associated with back-up pumping is based on the assumption that flooding will occur once the estimated protection level is exceeded. For example, for Option 2 it was assumed that flooding would occur once every ten years (total of ten times over the 100 year service life) and the cost to provide back-up pumping during each event would be $100,000 (approximately 50% of the maximum annual cost for pumping recorded in 1998).

TABLE 4.5
Estimation of Back-up Pumping Costs

Option	Approximate Level of Protection	Estimated Annual Cost of Back-up Pumping
2	~10 year	$10,000
2a	~25 year	$5,000
3	< 2 year	$30,000
4	25 year	$2,000
5	2 year	$15,000

Additional assumptions that were made in the Present Value calculations include:

- The service life of the project is 100 years (that is any system constructed to address flooding will be maintained and operated over 100 years).
- The interest rate is 7.25% and the inflation rate is 2%.

4.3.1.3 Summary of Technical and Economic Evaluation

Table 4.6 provides a summary of the technical and economic evaluation of the different options. The net benefit was calculated as the difference between the present value cost of the current pumping and the present value cost of implementing each option.

TABLE 4.6
Summary of Technical and Economic Evaluation

Option #	Description	Protection Level	Present Value Cost	Net Benefit
1	Current Practices	< 2 year	$1,150,000	-
2	Gravity	>10 year	$700,000	$450,000
2a	Gravity	<25 year	$650,000	$500,000
3	Pump to sewer	< 2 year	$1,840,000	($690,000)
4	Pump to ditch	~25 year	$1,320,000	($170,000)
5	Pump from Ditch	2 year	$920,000	$230,000

Technically only Options 2, 2a and 4 meet the goal of the study since they provide a level of reduction to the frequency of flooding.

Economically Option 2a would be preferred since it has the greatest benefit/loss ratio. The level of protection provided is approximately 1 in 25 years which is consistent with (or slightly greater than) other municipal drainage works designed using the City of Hamilton's standard drainage criteria. In addition, Option 2 or 2a would not preclude the future addition of a pump lift station if a higher level of service were required. Hence it is recommended that Option 2a be selected for implementation.

5 Lot Level Controls

As infill development proceeds in the Hamilton Beach Neighbourhood there is the potential for adjacent existing development to be impacted. The second goal of the Master Drainage Plan is to ensure that new development in the Hamilton Beach neighbourhood:

> *"...must not impact existing development due to changes in grade and drainage patterns."*

The hydrologic and hydraulic modeling that was completed for the Hamilton Beach neighbourhood indicates that there will be an increase in peak flows and runoff volumes due to the proposed redevelopment. The modeling also indicates that the storm sewers themselves will have the capacity to convey the additional flows. However, the existing outlets under the QEW are already at or near capacity at some locations. Therefore, it will be necessary to provide lot level control on post-development peak flows from some of the redeveloped lots (see requirements described in Section 5.2).

The City of Hamilton has site plan control for all redeveloped lots west (bay side) of Beach Boulevard. It is recommended that the City of Hamilton have site plan control east of Beach Boulevard as well since any redevelopment in this area has the potential to impact existing development. The site plan control provides a means to control lot grades during and after redevelopment.

5.1 Major Overland Flow Route

The major overland flow route for the Hamilton Beach neighbourhood is the streets. As previously discussed the general slope of the beach strip is from the east to the west (lake side to the bay side). However, Beach Boulevard tends to intercept overland flow from the land on the east side. Overland flows then follow the road grades to the west side of the beach strip where catchbasins and culverts convey the flows under the QEW to the harbour. Figures 3.1 and 3.2 show the overland flow route. To ensure that any impacts to existing development are mitigated it is important that future development in the Hamilton Beach neighbourhood make use of these existing overland flow routes.

5.2 Lot Level Drainage and Stormwater Management

The following sections of the report provide recommendations for grading and stormwater management for the different types of redevelopment for the entire Hamilton Beach neighbourhood.

5.2.1 Single Family Residential

The majority of redevelopment in the Hamilton Beach neighbourhood will be as single family residential lots. In general, these lots occur as groups of adjacent lots, however, there are a few proposed single infill lots as well. Figures 5.1, 5.2 and 5.3 provide an illustration of the grading required for the redevelopment of single family residential lots.

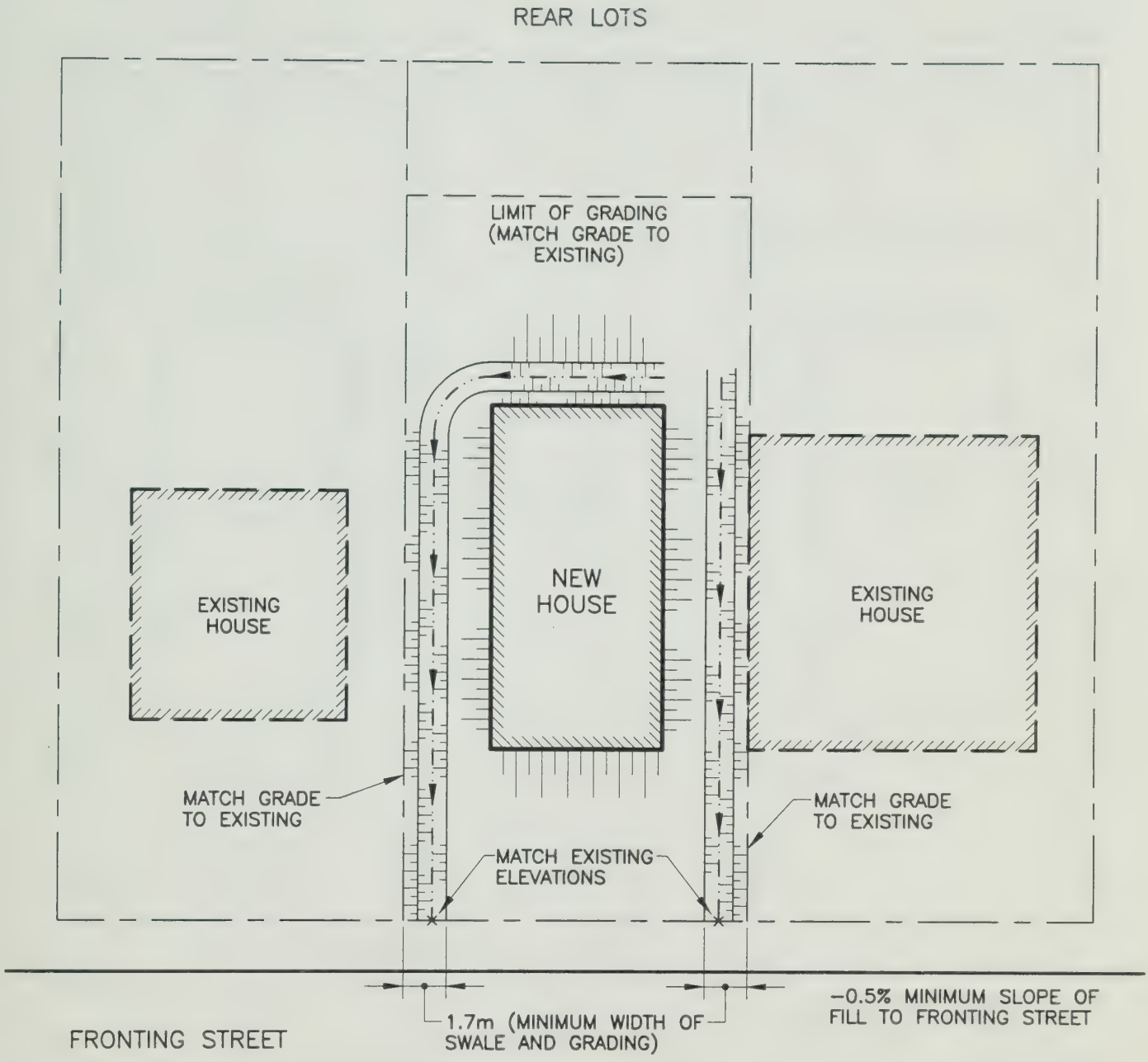
- Where single family residential infill development is proposed, the lots should be raised to an elevation such that the building envelope slopes to the fronting road (at a minimum grade of 0.5%). In most cases this will result in a lot elevation greater than 76.0 m (as required by by-law 98-281).
- For single infill lots a 1.7-m side yard on either side of the building envelope must be provided to allow for a drainage swale and provide for any grading to existing elevations. The raised building envelopes may only extend to 85% of the lot depth (unless otherwise specified). This includes any sloping required for matching to existing grades and a rear swale to convey runoff towards side swales, which convey flows to the front of the house. The remaining 15% of the lot length (from the rear lot boundary) is to remain at existing grade.
- For multiple adjoining infill lots where development proceeds simultaneously a common swale between the houses may be constructed. Each adjacent lot to be developed must provide a minimum allowance of 0.75-m for the swale. The end lots, which side onto an existing development, must provide a minimum allowance of 1.7-m on that side of the lot to provide for a swale and grading to existing elevations. Where possible a swale should be constructed along the rear lots to ensure a positive drainage route. If this is not possible approximately 15% of the lot length (from the rear lot boundary) should be maintained at existing grades.
- Where there is no positive drainage for the rear lots and grading changes may impact existing development the entire depth of the lot must be graded towards the road it fronts onto (or towards the road it sides onto if appropriate).
- Drainage swales must 1) be constructed to convey the peak flow from a 10 year storm; 2) have a minimum slope of 0.5%; 3) convey drainage from the building envelope to the fronting road. The drainage swales must not impact existing development.

It is recommended that the minimum lot size for a single infill redevelopment be 12 m in width (40 feet) to allow for the 1.7-m side yard required on either side of the building envelope for drainage. The minimum lot size for multiple lot redevelopment should be 11 m in width (35 feet) to allow for the 0.75-m and 1.7-m side yards required on either side of the building envelope for drainage. Figures 5.4 and 5.5 illustrate which lots could not be developed given the above lot size constraints.

It is further recommended that all roof leaders be directed to pervious surfaces (such as the rear yard) to allow for infiltration of stormwater. Roof leaders must not be directly connected to the storm sewer.

5.2.2 Attached Housing and Low Density Apartments

The main node of future attached housing and low-density apartments is located west of Beach Boulevard, south of Arden Avenue and north of Bayside Avenue. These types of developments tend to have parking lots associated with them and a higher imperviousness (≥ 45%) than single family residential lots. Since there is more



CLIENT

CITY OF HAMILTON

TITLE

SINGLE INFILL DEVELOPMENT TYPICAL GRADING

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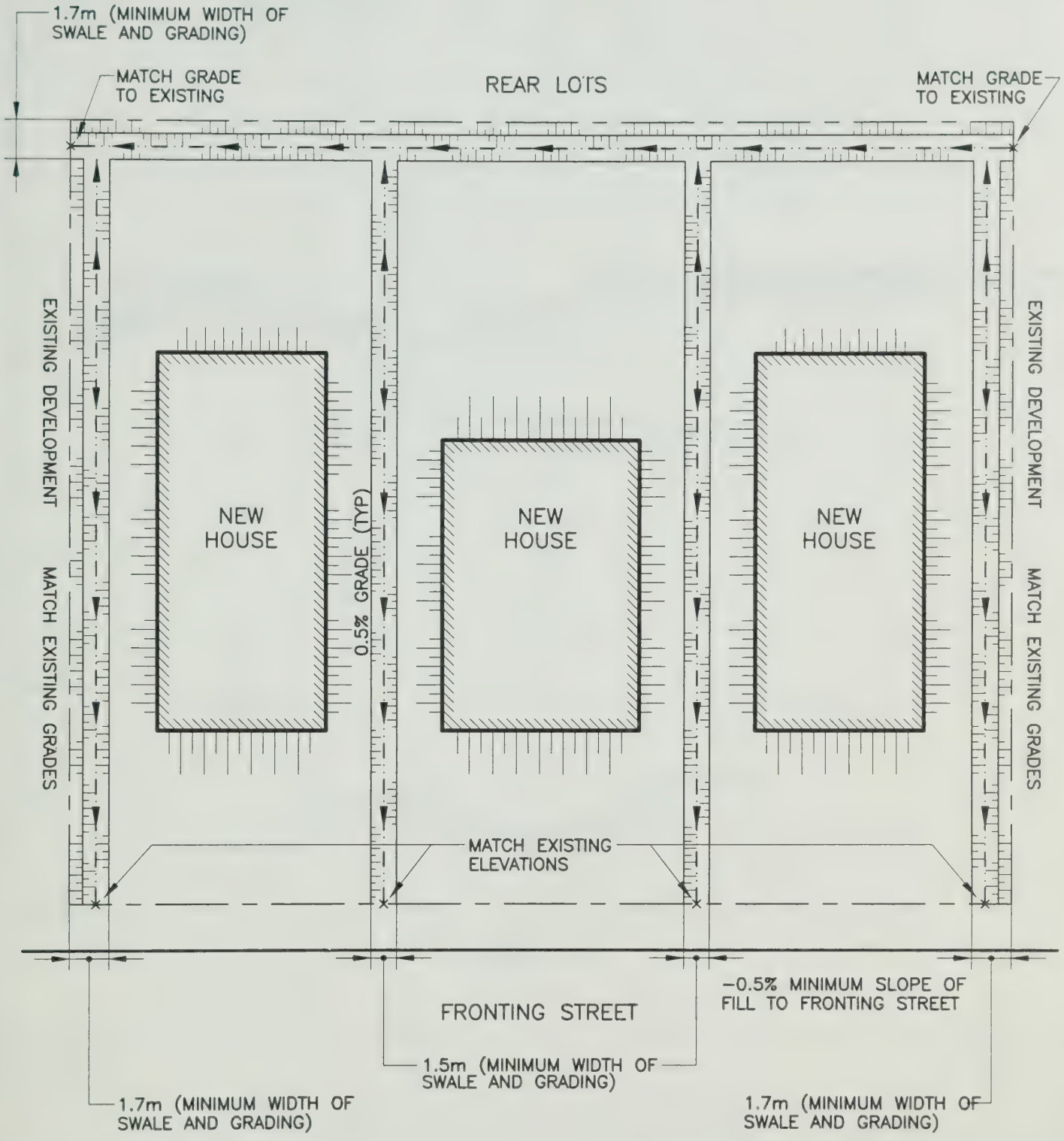
Date JULY 29 1999	Proj. No. 1499015-01-101
Scale N.T.S.	Figure No. 5.1 — Gr.No. 01



CLIENT

CITY OF HAMILTON

TITLE

MULTIPLE INFILL DEVELOPMENT TYPICAL GRADING (WHERE REAR SWALE IS POSSIBLE)

Marshall Macklin Monaghan

Consulting Engineers . Surveyors . Planners
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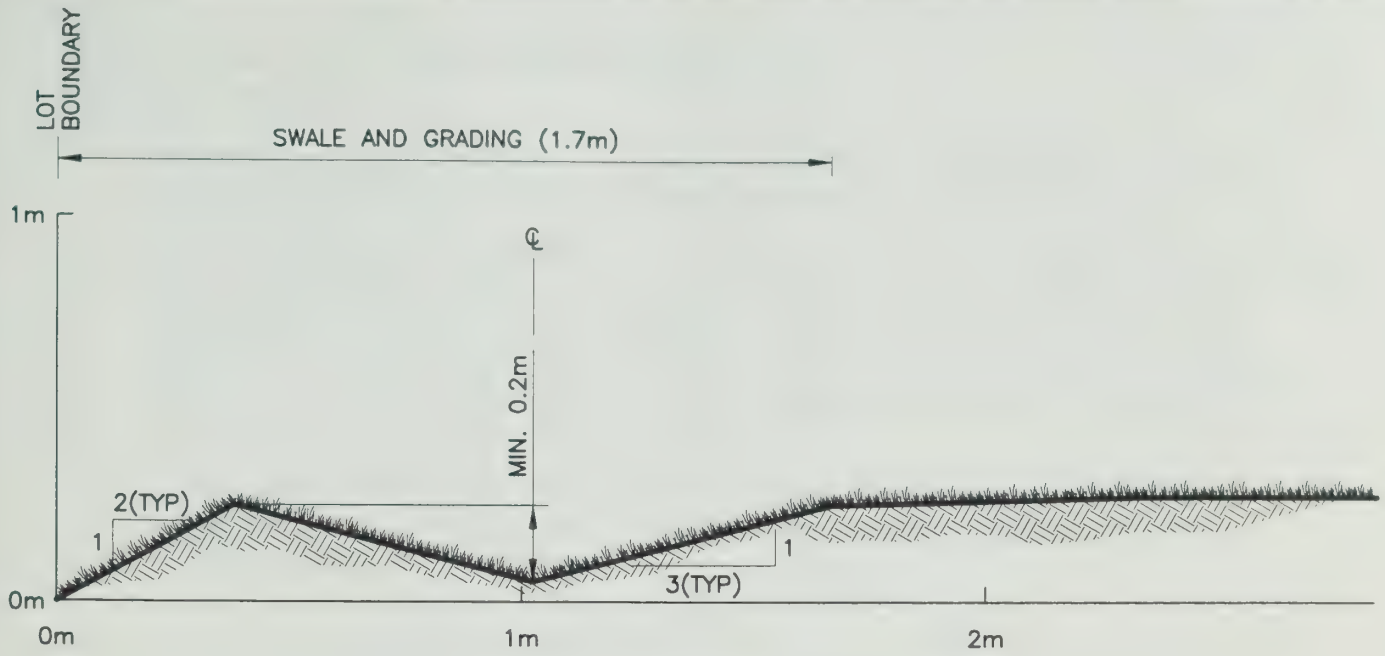
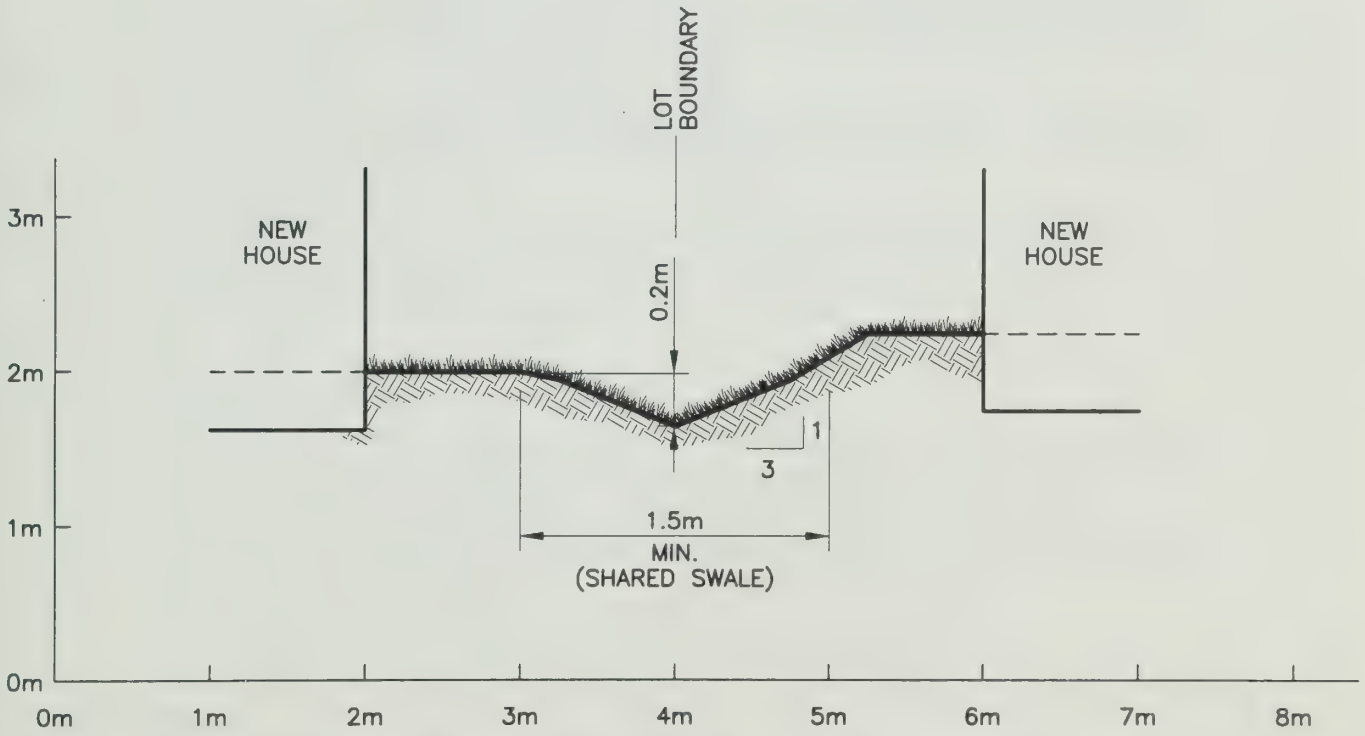
Tele (905)882-1100 Fax (905)882-0055

Checked J.C.P.		Drawn AutoCAD/B.K.B.	
Date JULY 29 1999		Proj. No. 1499015-01-101	
Scale N.T.S.		Figure No. 5.2	Gr.No. 01



TYPICAL SIDELOT SWALE ADJACENT TO EXISTING DEVELOPMENT



TYPICAL SWALE BETWEEN TWO NEW DEVELOPMENTS

CLIENT

CITY OF HAMILTON

TITLE

SWALES

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Checked J.C.P.	Drawn AutoCAD/B.K.B.
Date JULY 29 1999	Proj. No. 1499015-01-101
Scale N.T.S.	Figure No. 5.3 Gr.No. 01

opportunity to provide storage, on parking surfaces, it is recommended that post-development flows up to the 100-year storm be controlled to pre-development levels.

As with the single family residential developments, it is important that the overland flow is directed to a major overland flow route and does not impact existing development. Lots that back onto the QEW may provide an overland flow route directly to the ditch along the QEW noise berm. Those lots that do not back onto the QEW must be graded, with drainage swales similar to those proposed for the single-family residential lots, to the fronting street.

It is recommended that all roof leaders be directed to pervious surfaces (such as landscaped areas) to allow for infiltration of stormwater. Roof leaders must not be directly connected to the storm sewer.

5.2.3 Commercial and Civic/Institutional

Lands slated for commercial and institutional development occur throughout the Hamilton Beach neighbourhood. These developments tend to have a higher imperviousness ($\geq$ 60%) than single family residential lots. Since there is an opportunity to provide stormwater storage, on parking surfaces and potentially rooftops, it is recommended that post-development flows up to the 100-year storm be controlled to pre-development levels.

All overland flows from the developments must be directed to a street, with the exception of any proposed developments that back onto the QEW. These lots are to be graded towards the QEW.

It is recommended that all roof leaders be directed to pervious surfaces (such as landscaped areas) to allow for infiltration of stormwater. Roof leaders must not be directly connected to the storm sewer.

5.2.4 Public Parking Lots

Two public parking lots are proposed at the south end of the Hamilton Beach neighbourhood, west of Beach Boulevard and south of Fletcher Avenue. It is recommended that the parking lots control post-development peak flows to pre-development levels (up to the 1 in 100-year storm) to ensure that there are no impacts to the existing storm sewer.

The parking lots should be graded to the west towards the QEW ditch.

5.2.5 Parks

It is recommended that redevelopment of lots as parks take into consideration the natural grades and drainage directions. Minimal alteration to the grades should be made to ensure that any natural ponding characteristics are maintained.

EASTPORT
QUEEN
DRIVE
BEACH
MANOR AVENUE
BOULEVARD
THIRD AVENUE
FOURTH AVENUE
ELIZABETH
EASTPORT
DRIVE
HAMILTON HARBOUR

LEGEND

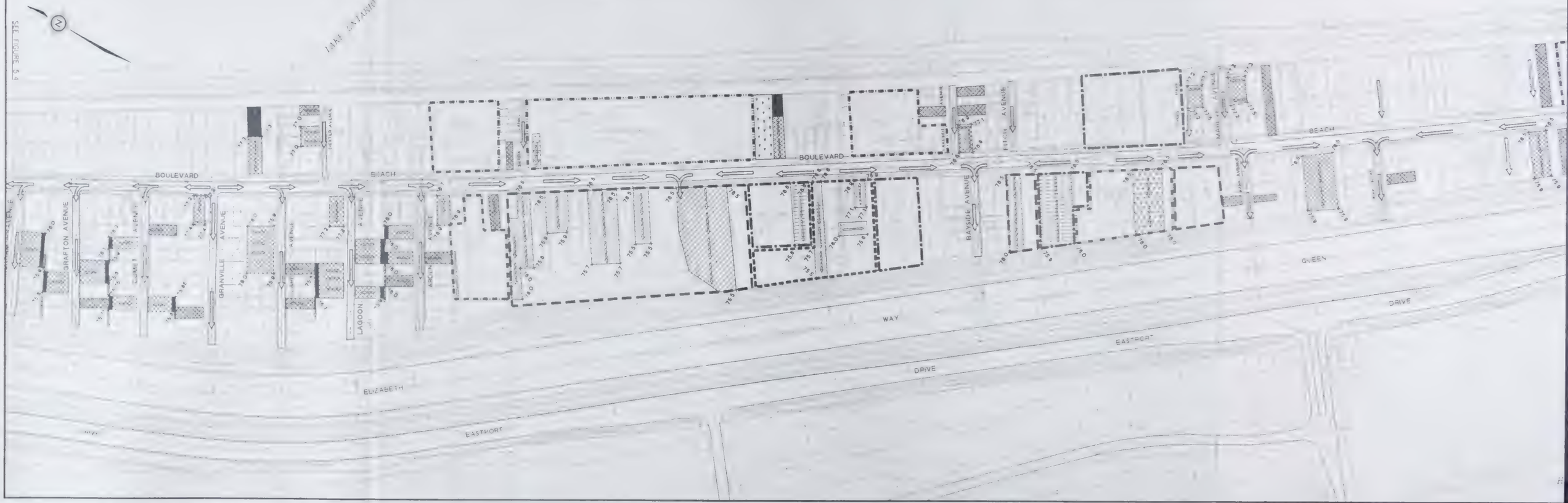
- RESIDENTIAL – EXISTING
- SINGLE FAMILY RESIDENTIAL – PROPOSED
- ATTACHED HOUSING – EXISTING
- ATTACHED HOUSING – PROPOSED
- LOW DENSITY APARTMENTS – EXISTING
- LOW DENSITY APARTMENTS – PROPOSED
- INSTITUTIONAL – EXISTING
- INSTITUTIONAL – PROPOSED
- MIXED USE – EXISTING
- COMMERCIAL – PROPOSED
- PARK – EXISTING
- PARK – PROPOSED
- PARKING LOT – PROPOSED
- STRIP OF LAND TO REMAIN @ EXISTING GRADE ~15% OF LOT LENGTH
- ×76.0 APPROX. ELEVATION CONTROL
- [illegible]
- MAJOR OVERLAND FLOW ROUTE
- [illegible]
- [illegible] PROVIDES OFF-MAIN STORAGE)

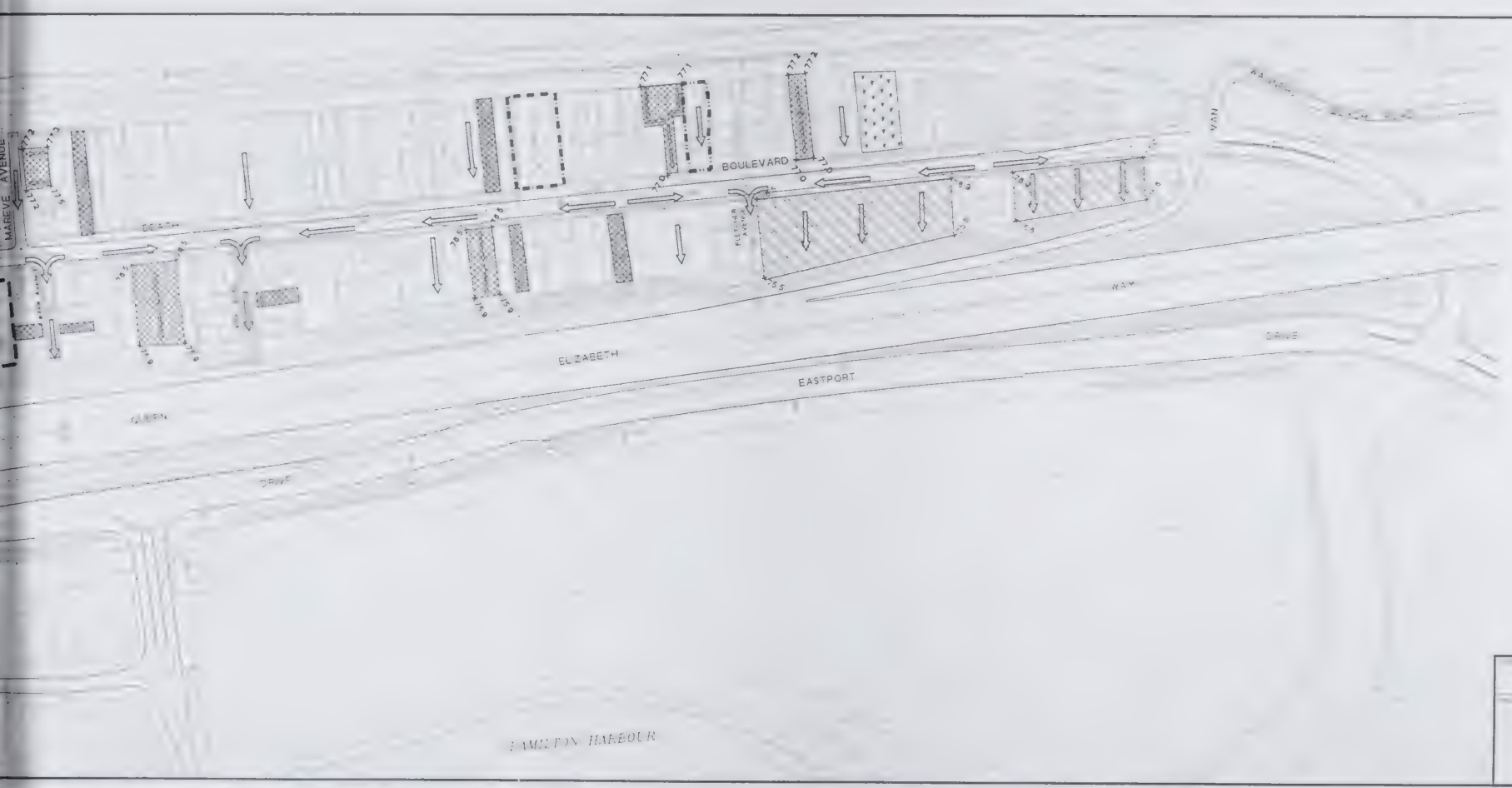
CITY OF HAMILTON

LOT GRADING MASTER DRAINAGE PLAN

Marshall Macklin Monaghan

Checked J.P.	Drawn A.M.D.C.
Date JULY 29 1999	Proj. No 1499015-01-101
Scale AS SHOWN	Figure No. 5.4


SEE FIGURE 5.4
LAKE ONTARIO
BOULEVARD
BEACH
GRAFTON AVENUE
GRANVILLE AVENUE
LAGOON
BAYSIDE AVENUE
FETCH AVENUE
MAPLE AVENUE
ELIZABETH
EASTPORT
WAY
DRIVE
GREEN



LEGEND

- RESIDENTIAL – EXISTING
- SINGLE FAMILY RESIDENTIAL – PROPOSED
- ATTACHED HOUSING – EXISTING
- LOW DENSITY APARTMENTS – EXISTING
- LOW DENSITY APARTMENTS – PROPOSED
- CIVIC/INSTITUTIONAL – EXISTING
- COMMERCIAL – EXISTING
- PARK – EXISTING
- PARK – PROPOSED
- PARKING LOT – PROPOSED

CITY OF HAMILTON

LOT GRADING MASTER DRAINAGE PLAN

Marshall Macklin Monaghan

Proj. No. 1499215-01-101

5.5

6 Conclusions

6.1 Street Flooding

Six different options were investigated to alleviate the existing chronic flooding of streets in the Hamilton Beach neighbourhood.

Option 1 (Continue Current Practices): This option includes non-construction means to improve drainage to the outlets under the QEW berm. It is recommended to remove any earthen or wooden berms at the street ends, which obstruct the flow of water to the QEW ditch and to ensure outlets are free of sediment, vegetation and garbage.

Option 2 (Gravity Drainage Improvements): Convey stormwater by gravity to a new outlet under the QEW. Provide ditch inlet catchbasins at the end of Wickham, Grafton, Comet, Granville, Clare and Arden and collect stormwater in a pipe running parallel to the QEW noise berm. A new outlet pipe at the end of Granville Avenue will convey stormwater across the QEW to the ditch between Eastport Drive and the QEW. The pipe will be large enough to ensure peak flows will be conveyed under low head conditions due to high lake levels.

Option 2a (Gravity Drainage Improvements with larger storm sewers): Similar to Option 2 but storm sewer sizes were increased to the next larger pipe diameter.

Option 3 (Pumping to Existing Sewer): Convey stormwater to an outlet by pumping to an existing storm sewer. Provide catchbasins at the end of Windemere, Knapmans, Wickham, Grafton, Comet, Granville, Clare and Arden and collect stormwater in a pipe running parallel to the QEW noise berm. A lift station at Knapmans Drive would pump stormwater to an existing storm sewer that runs between Eastport Drive and the QEW. Different pump rates were considered to determine the impact pumping will have on the downstream storm sewer. This option may include a new pipe under the QEW.

Option 4 (Pumping to Existing Ditch): Convey stormwater to an outlet by pumping to an existing ditch between Eastport Drive and the QEW. Similar to Option 3 except the lift station will be located at the end of Granville Avenue and water will be pumped to the existing ditch. Different pump rates were reviewed to determine the impact pumping will have on the downstream ditch. This option may include a new pipe under the QEW.

Option 5 (Lower Tailwater by Pumping Existing Ditch): Water levels in the ditch located between the QEW and Eastport Drive would be kept low by installing a pump lift station at the downstream end of the ditch (opposite Kirk Road). By lowering the water level in the ditch, storm sewer outlets from the Hamilton Beach Neighbourhood would not be submerged.

The options were evaluated technically through the use of a hydrologic (OTTSWMM) and hydraulic (EXTRAN) model. The options were also evaluated economically using Present Value costs over a service life of 100 years. The following table provides a summary of the results of that evaluation.

Summary of Technical and Economic Evaluation

Option #	Description	Protection Level	Present Value	Net Benefit
1	Current Practices	< 2 year	$1,150,000	-
2	Gravity	>10 year	$700,000	$450,000
2a	Gravity	<25 year	$650,000	$500,000
3	Pump to sewer	< 2 year	$1,840,000	($690,000)
4	Pump to ditch	~25 year	$1,320,000	($170,000)
5	Pump from Ditch	2 year	$920,000	$230,000

Technically only Options 2, 2a and 4 meet the goal of the study since they provide a level of reduction to the frequency of flooding.

Economically Option 2a would be preferred since it has the greatest benefit/loss ratio. The level of protection provided is approximately 1 in 25 years which is consistent with (or slightly greater than) other municipal drainage works designed using the City of Hamilton's standard drainage criteria. In addition, Option 2 or 2a would not preclude the future addition of a pump lift station if a higher level of service were required. Hence it is recommended that Option 2a be selected for implementation.

6.2 *Lot Grading*

The City of Hamilton has site plan control for all redeveloped lots west (bay side) of Beach Boulevard. It is recommended that the City of Hamilton have site plan control east of Beach Boulevard as well since any redevelopment in this area has the potential to impact existing development. The site plan control provides a means to control lot grades during and after redevelopment.

A Master Drainage Plan for the overland flow has been prepared and drainage criteria specified for redevelopment of lots designated for Single Family Residential, Attached Housing, Low Density Apartments, Commercial, Civic Institutional, Parking Lots and Parks. Major overland flow routes and control elevations for redevelopment are shown on Figures 5.4 and 5.5. The stormwater management criteria for redevelopment are provided in Section 5.2 of the preceding text.

APPENDIX

1

PROGRAM HYDSTAT OUTPUT

INPUT NVAR = 2
MULT = 0 IAV = 0

COEFF1 = 1.000000
COEFF2 = 1.000000
EX1 = 1.000
EX2 = 1.000
CONST = .00000

LAKE ONTARIO ANNUAL MAX INSTANTANEOUS WATER LEVELS

NUMBER OF POINTS= 0 DISTRIBUTION TYPE = 6 RESULTS TO BE WRITTEN = 1

XLO = .43111E+01 XSC = .88180E-03 LOWER BOUND OF X = .11790E+02

1

LAKE ONTARIO ANNUAL MAX INSTANTANEOUS WATER LEVELS

SUMMARY TABLE
****

	DISTRIBUTION TYPE	LOCATION PARAMETER	SCALE PARAMETER	SHAPE PARAMETER	LEAST SQUARES STANDARD ERROR	5 YR ESTIMATE	100 YR ESTIMATE
1	NORMAL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
2	LOG-NORMAL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
3	GUMBEL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
4	LOG-GUMBEL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
5	PEARSON-3	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
6	LOG PEARSON-3	.43111E+01	.88180E 03	.11790E+02	.00000E+00	75.484	75.930
7	LOG-NORMAL-3	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
8	GEN. EXTREME V	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000

MINIMUM CHI-SQUARE DISTRIBUTION NO.= 6
MINIMUM LEAST-SQUARES DISTRIBUTION NO.= 6

1

PROGRAM HYDSTAT OUTPUT

CURRENT NVAR = 2
MULT = 0 IAV = 0

COEFF1 = 1.000000

COEFF2 = 1.000000
EX1 = 1.000
EX2 = 1.000
CONST = .00000

FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

NUMBER OF POINTS= 0 DISTRIBUTION TYPE = 3 RESULTS TO BE WRITTEN = 1

XLO = .12640E+00 XSC = .55100E-01 LOWER BOUND OF X = .00000E+00

1

FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

SUMMARY TABLE
****

	DISTRIBUTION TYPE	LOCATION PARAMETER	SCALE PARAMETER	SHAPE PARAMETER	LEAST-SQUARES STANDARD ERROR	5 YR ESTIMATE	100 YR ESTIMATE
1	NORMAL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
2	LOG-NORMAL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
3	GUMBEL	.12640E+00	.55100E-01	.00000E+00	.00000E+00	.209	.380
4	LOG-GUMBEL	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
5	PEARSON-3	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
6	LOG-PEARSON-3	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
7	LOG-NORMAL-3	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000
8	GEN. EXTREME V	.00000E+00	.00000E+00	.00000E+00	.00000E+00	.000	.000

MINIMUM CHI-SQUARE DISTRIBUTION NO.= 3
MINIMUM LEAST-SQUARES DISTRIBUTION NO.= 3

SUMMARY OF MINIMUM LEAST-SQUARES DISTRIBUTIONS

MINIMUM LEAST-SQUARES DISTRIBUTION NUMBER FOR PREVIOUS VARIABLE=6
MINIMUM LEAST-SQUARES DISTRIBUTION NUMBER FOR CURRENT VARIABLE=3

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

CUMULATIVE DISTRIBUTION CDF(Z)	COMPUTED Z VALUE Z(I)
.0052	74.9119
.0058	74.9399
.0075	74.9679
.0108	74.9958

.0162	75.0218
[illegible]	[illegible]
.0356	75.0797
.0505	75.1077
.0695	75.1356
[illegible]	[illegible]
1305	75. 1916
.1525	75.2195
.1887	75.2475
[illegible]	[illegible]
.2714	75. 3034
3168	75. 3314
[illegible]	[illegible]
.4122	75.3873
[illegible]	[illegible]
.5085	75.4432
.5554	75.4712
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
.7558	75.6150
[illegible]	[illegible]
.8160	75.6669
[illegible]	[illegible]
.8644	75.7229
[illegible]	[illegible]
.9022	75.7788
[illegible]	[illegible]
[illegible]	[illegible]
.9433	75.8627
.9519	75.8906
.9603	75.9186
[illegible]	[illegible]
[illegible]	[illegible]
.9779	76.0025
.9820	76.0304
.9854	76.0584
.9882	76.0864
[illegible]	[illegible]
.9919	76.1423
.9940	76.1703
[illegible]	[illegible]
[illegible]	[illegible]
.9946	76.2541
[illegible]	[illegible]

CORRELATION BETWEEN PREVIOUS VARIABLE AND
LEVEE DEPTH FOR EITHER PERIOD CROPS: Proposed DEVELOPMENT

THE NORMAL CUMULATIVE DISTRIBUTION (NO. 1)

(XLO = 75.45776 XSC = .22882)

ORDER I	CUMULATIVE DISTRBUTION CDF(X)	X(I)
1	.1000	75.1645
2	.2000	75.2652
3	.3000	75.3379
4	.4000	75.3999
5	.5000	75.4578
6	.6000	75.5156
7	.7000	75.5777
8	.8000	75.6503
9	.9000	75.7510
10	.9600	75.8584
11	.9800	75.9278
12	.9900	75.9902
13	.9950	76.0472
14	.9980	76.1164

LEAST SQUARES STANDARD ERROR = .25794E-02

COMPUTED CHI-SQUARE= 1.64000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE LOG-NORMAL CUMULATIVE DISTRIBUTION (NO .2)

(XLO = 4.32357 XSC = .00303)

ORDER I	CUMULATIVE DISTRBUTION CDF(X)	X(I)
1	.1000	75.1649
2	.2000	75.2652
3	.3000	75.3377
4	.4000	75.3996
5	.5000	75.4574
6	.6000	75.5153
7	.7000	75.5773
8	.8000	75.6501
9	.9000	75.7511
10	.9600	75.8589
11	.9800	75.9286
12	.9900	75.9914
13	.9950	76.0488
14	.9980	76.1185

LEAST SQUARES STANDARD ERROR = .25448E-02

COMPUTED CHI-SQUARE= 1.64000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE GUMBEL CUMULATIVE DISTRIBUTION (NO.3)

(PARAMETERS XLO= 75.354790 XSC= .178407)

ORDER (I)	CUMULATIVE DISTRIBUTION CDF(X)	X(I)
1	.1000	75.2060
2	.2000	75.2699
3	.3000	75.3217
4	.4000	75.3704
5	.5000	75.4202
6	.6000	75.4746
7	.7000	75.5387
8	.8000	75.6224
9	.9000	75.7563
10	.9600	75.9254
11	.9800	76.0509
12	.9900	76.1755
13	.9950	76.2996
14	.9980	76.4633

LEAST SQUARES STANDARD ERROR = .27434E-02

COMPUTED CHI-SQUARE= 3.62000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE LOG-GUMBEL CUMULATIVE DISTRIBUTION (NO. 4)

(PARAMETERS XLO= 4.322205 XSC= .002363)

ORDER (I)	CUMULATIVE DISTRIBUTION CDF(X)	X(I)
1	.1000	75.2062
2	.2000	75.2699
3	.3000	75.3215
4	.4000	75.3701
5	.5000	75.4199
6	.6000	75.4743

7	.7000	75.5384
8	.8000	75.6221
9	.9000	75.7563
10	.9600	75.9262
11	.9800	76.0525
12	.9900	76.1781
13	.9950	76.3034
14	.9980	76.4691

LEAST SQUARES STANDARD ERROR = .27763E-02

COMPUTED CHI SQUARE= 3.62000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE PEARSON-III CUMULATIVE DISTRIBUTION (NO.5)
(XM = 75.45776 XS = .52356570E-01 CS = .41861)

XLO = .74365E+02 XSC = .47892E-01 XSH = .22827E+02

ORDER (1)	CUMULATIVE DISTRIBUTION CDF(X)	X(I)
1	.1000	75.1768
2	.2000	75.2622
3	.3000	75.3274
4	.4000	75.3856
5	.5000	75.4419
6	.6000	75.5002
7	.7000	75.5650
8	.8000	75.6440
9	.9000	75.7592
10	.9600	75.8894
11	.9800	75.9776
12	.9900	76.0598
13	.9950	76.1374
14	.9980	76.2346

LEAST SQUARES STANDARD ERROR = .10808E-02

COMPUTED CHI SQUARE= .32000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE LOG-PEARSON-III CUMULATIVE DISTRIBUTION (NO.6)
(XM = 4.32357 XS = .91837740E-05 CS = .41091)

XLO = .43088E+01 XSC = .62262E-03 XSH = .23690E+02

ORDER (1)	CUMULATIVE DISTRIBUTION CDF(X)	X(I)
1	.1000	75.1768
2	.2000	75.2623
3	.3000	75.3274
4	.4000	75.3856
5	.5000	75.4418
6	.6000	75.5001
7	.7000	75.5649
8	.8000	75.6439
9	.9000	75.7591
10	.9600	75.8894
11	.9800	75.9778
12	.9900	76.0602
13	.9950	76.1381
14	.9980	76.2356

LEAST SQUARES STANDARD ERROR = .10794E-02

COMPUTED CHI-SQUARE= .32000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

THE 3 PARAMETER LOG-NORMAL CUMULATIVE DISTRIBUTION (NO.7)

(XLO = .49147 XSC = .13799)

ORDER I	CUMULATIVE DISTRBUTION CDF(X)	X(I)
1	.1000	73.3774
2	.2000	73.4632
3	.3000	73.5283
4	.4000	73.5863
5	.5000	73.6424
6	.6000	73.7004
7	.7000	73.7649
8	.8000	73.8436
9	.9000	73.9586
10	.9600	74.0892
11	.9800	74.1781
12	.9900	74.2613
13	.9950	74.3402
14	.9980	74.4395

LEAST SQUARES STANDARD ERROR = .17921E+00

COMPUTED CHI-SQUARE= 1000.00000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

GEV (MOMENTS) : A = .210 U = 75.365
K = .155 B = .7672E+02

PARAMETERS OF THE GENERALIZED EXTREME VALUE DIST (NO. 8)

(UNBIASED MOMENT ESTIMATORS)

MEAN VALUE OF X = 75.36474 VARIANCE OF X = .21004400E+00 COEF.OF SKEWNESS= .15460

1	.1000	75.1778
2	.2000	75.2610
3	.3000	75.3252
4	.4000	75.3830
5	.5000	75.4396
6	.6000	75.4987
7	.7000	75.5649
8	.8000	75.6459
9	.9000	75.7640
10	.9600	75.8948
11	.9800	75.9801
12	.9900	76.0562
13	.9950	76.1242
14	.9980	76.2035

LEAST SQUARES STANDARD ERROR = .96133E-03

COMPUTED CHI-SQUARE= .10000

1

COMBINATION BETWEEN PREVIOUS VARIABLE AND
FLOOD DEPTHS FOR RETURN PERIOD STORMS - Proposed DEVELOPMENT

SUMMARY TABLE
****

	DISTRIBUTION TYPE	LOCATION PARAMETER	SCALE PARAMETER	SHAPE PARAMETER	LEAST-SQUARES STANDARD ERROR	5 YR ESTIMATE	100 YR ESTIMATE
1	NORMAL	.75458E+02	.22882E+00	.00000E+00	.25794E-02	75.650	75.990
2	LOG-NORMAL	.43236E+01	.30305E-02	.00000E+00	.25448E-02	75.650	75.991
3	GUMBEL	.75355E+02	.17841E+00	.00000E+00	.27434E-02	75.622	76.175
4	LOG-GUMBEL	.43222E+01	.23629E-02	.00000E+00	.27763E-02	75.622	76.178
5	PEARSON 3	.74365E+02	.47892E-01	.22827E+02	.10808E 02	75.644	76.060
6	LOG-PEARSON-3	.43088E+01	.62262E-03	.23690E+02	.10794E-02	75.644	76.060
7	LOG NORMAL 3	.49147E+00	.13799E+00	.00000E+00	.17921E+00	73.844	74.261
8	GEN. EXTREME V	.75365E+02	.21004E+00	.15460E+00	.96133E-03	75.646	76.056

MINIMUM CHI-SQUARE DISTRIBUTION NO.= 8

MINIMUM LEAST-SQUARES DISTRIBUTION NO.= 8

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